SOVIET HYDRO ENGINEERING

A Classified Collection of Research Reports

TRANSLATED FROM RUSSIAN

Published for the U.S. Department of the Interior

and the National Science Foundation, Washington D.C.

by the Israel Program for Scientific Translations

Ministerstvo Stroitel'stva Elektrostantsii SSSR. Glavenergoproekt.
Vsesoyuznyi Nauchno-Issledovatel'skii Institut Gidrotekhniki im. B. E. Vedeneev

Ministry for the Construction of Electric Power Plants of the U.S.S.R.
Department for the Design of Power Engineering Structures
All-Union Scientific Research Institute of Hydraulic Engineering im. B. E. Vedeneev

SOVIET HYDRO ENGINEERING

A Classified Collection of Research Reports

(Annotatsii zakonchennykh v 1958 godu nauchno-issledovatel'skikh rabot po gidrotekhnike)

GEI
Gosudarstvennoe Energeticheskoe Izdatel'stvo
Moskva-Leningrad
1959

Translated from Russian

Israel Program for Scientific Translations
Jerusalem 1964

OTS 63-11159

Published Pursuant to an Agreement with
THE U. S. DEPARTMENT OF THE INTERIOR
and
THE NATIONAL SCIENCE FOUNDATION, WASHINGTON D. C.

IPST Cat. No. 880

Translated by A. Wald and L. Pommerantz

Printed in Jerusalem by Sivan Press

Binding K. Wiener

Available from the Office of Technical Services
U. S. Department of Commerce, Washington 25, D. C.

XI/5/1.5

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EXPLANATORY LIST OF ABBREVIATED NAMES OF U.S.S.R. INSTITUTIONS, ORGANIZATIONS, ETC., APPEARING IN THIS TEXT

Abbreviation	Full name (transliterated)	Translation
AN SSSR	Akademiya Nauk SSSR	Academy of Sciences of the U.S.S.R.
ArmNIIG	Armyanskii Nauchno-Issledovatel'skii Institut Gidrotekhniki	Armenian Scientific Research Institute of Hydraulic Engineering
ASiA	Akademiya Stroitel'stva i Arkhitektury SSSR	Academy of Construction (Building) and Architecture of the U.S.S.R.
AzNIIGiM	Nauchno-Issledovatel'skii Institut Gidrotekhniki i Melioratsii Ministerstva Vodnogo Khozyaistva Azerbaidzhanskoi SSR	Scientific Research Institute of Hydraulic Engineering and Reclamation of the Ministry of Water Management of the Azerbaijani S.S.R.
Bakgidep	Bakinskii Gosudarstvennyi Proektnyi Institut	Baku State Design Institute
BPI	Belorusskii Politekhnicheskii Institut	The Belorussian Polytechnical Institute
Bratsgesstroi	Stroitel'stvo Bratskoi Gidroelektrostantsii	Main Administration for the Construction of the Bratsk HEP
Dneproges	Dneprovskaya Gidroelektrostantsiya imeni V.I. Lenin	Dnieper HEP imeni V.I. Lenin
Donbasskanalstroi	Glavnoe Upravlenie po Stroitel'stvu Donbasskogo Kanala	Main Administration for the Construction of the Donbass Canal
EGDA	Metod Elektrogidrodinamicheskoi Analogii	Electrohydrodynamic-analogy Method
ENIIZiM	Estonskii Nauchno-Issledovatel'skii Institut Zemledeliya i Melioratsii	Estonian Scientific Research Institute of Agriculture and Reclamation
Gidep	Vsesoyuznyi Gosudarstvennyi Proektnyi Institut	All-Union State Design Institute

Abbreviation	Full name (transliterated)	Translation
Gidroenergoproekt	Vsesoyuznyi Trest po Proektirovaniyu Gidroelektrostantsii i Gidrouzlov	All-Union Trust for the Design and Planning of Hydro-Power Plants and Developments
Gidroproekt	Gosudarstvennyi Institut po Proektirovaniyu Gidrotekhnicheskikh Sooruzhenii	State Institute for Design and Planning of Hydro Structures
Gidrorybproekt	Gosudarstvennyi Institut po Proektirovaniyu Gidrotekhnicheskikh, Rybovodno-meliorativnykh i Prudovykh Sooruzhenii	State Institute for the Design and Planning of Hydrotechnical, Reclamation, Fish Breeding, and Fish-pond Structures
Gidrospetsfundamentstroi	Gosudarstvennyi Spetsializirovannyi Trest po Stroitel'stvu Fundamentov Gidrosooruzhenii	State Specialized Trust for the Erection of Foundations for Hydro Structures
Gidrospetstroi	Stroitel'stvo Spetsial'nykh Gidrosooruzhenii	Main Administration for the Construction of Special Hydro Structures
Gidrotsement	Vsesoyuznyi Gosudarstvennyi Nauchno-Issledovatel'skii i Proektnyi Institut Tsementnoi Promyshlennosti	All-Union Scientific Design Institute for the Cement Industry
Giprovodkhoz	Gosudarstvennyi Institut po Proektirovaniyu Vodokhozyaistvennogo i Meliorativnogo Stroitel'stva	State Institute for the Design and Planning of Water Management and Melioration Structures
GISI imeni V. P. Chkalova	Gor'kovskii Inzhenerno-Stroitel'nyi Institut imeni V. P. Chkalova	The Gor'kii Civil Engineering Institute imeni V. P. Chkalov
Glavgidroenergostroimontazh	Glavnoe Upravlenie po Stroitel'stvu i Montazhu Gidroenergeticheskikh Sooruzhenii	Main Administration for the Construction and Installation of Hydroelectric Structures
Glavleningradstroi	Glavnoe Upravlenie po Stroitel'stvu Leningradskogo Gorispolkoma	Main Administration for Building and Construction of the Leningrad Town Council

Abbreviation	Full name (transliterated)	Translation
Gosstroi	Gosudarstvennyi Komitet po Delam Stroitel'stva	Government Committee for Construction
GOST	Gosudarstvennyi Obshcheso-yuznyi Standart	All-Union Government Standard
GruzNIIGM	Gruzinskii Nauchno-Issledo-vatel'skii Institut Gidro-tekhniki i Melioratsii	Georgian Scientific Research Institute of Hydraulic Engineering and Land Reclamation
IEVKh AN Kirg.SSR	Institut Energetiki i Vodnogo Khozyaistva AN Kirgizskoi SSR	Institute of Power Engineering and Water Management of the Kirgiz SSR
IGiG AN Ukr.SSR	Institut Gidrologii i Gidro-tekhniki Ukr.SSR	Institute of Hydrology and Hydraulic Engineering of the Academy of Sciences of the Ukrainian SSR
InEG AN Arm.SSR	Institut Energetiki i Gidrav-liki AN Arm. SSR	Institute of Power Engineering and Hydraulics of the Academy of Sciences of the Armenian SSR
KIIVKh	Kievskii Institut Inzhenerov Vodnogo Khozyaistva	Kiev Institute of Water Management
Lenergo	Leningradskoe Raionnoe Up-ravlenie Energeticheskogo Khozyaistva	Leningrad District Administration for Power Engineering and Power Construction
Lengidep	Leningradskii Gosudarstven-nyi Proektnyi Institut	Leningrad State Design Institute
Lenotdelstroi	Leningradskoe Otdelenie Spetsial'nogo Stroitel'-stva	Leningrad Branch of the Agency for Special Constructions
LGMI	Leningradskii Gidrometeo-rologicheskii Institut	Leningrad Institute of Hydro Meteorology
LMZ	Leningradskii Metallicheskii Zavod	Leningrad Metal Works
MES SSSR	Ministerstvo Elektrostantsii SSSR	Ministry of Electric Power Plants of the U.S.S.R.

Abbreviation	Full name (transliterated)	Translation
MIIVKh imeni V.R. Vil'yamsa	Moskovskii Institut Inzhenerov Vodnogo Khozyaistva imeni V.R. Vil'yamsa	Moscow Institute of Water Engineers imeni V.R. Vil'yams
Minstroi	Ministerstvo Stroitel'stva	Ministry of Building and Construction
MISI imeni V.V. Kuibysheva	Moskovskii Ordena Trudovogo Krasnogo Znameni Inzhenerno-Stroitel'nyi Institut imeni V.V. Kuibysheva	The Moscow "Order of the Red Labor Banner" Civil Engineering Institute imeni V.V. Kuibyshev
Mosenergo	Moskovskoe Raionnoe Upravlenie Energeticheskogo Khozyaistva	Moscow District Administration for Power Engineering and Power Construction
Mosgidep	Moskovskii Gosudarstvenii Proektnyi Institut	Moscow State Design Institute
Mosgiprotrans	Moskovskii Gosudarstvenii Proektnyi Institut Transportnogo Stroitel'stva	Moscow State Design Institute for Transportation Construction
Mosmetrostroi	Upravlenie po Stroitel'stvu Moskovskogo Metropolitena	Main Administration for the Construction of the Moscow Metro
MSES	Ministerstvo Stroitel'stva Elektrostantsii	Ministry for the Construction of Electric Power Plants
MSPMKhP	Ministerstvo Stroitel'stva Predpryatii Metallurgicheskoi i Khimicheskoi Promyshlennosti	Ministry for the Construction of Plants of the Metallurgical and Chemical Industry
MVO SSSR	Ministerstvo Vysshego Obrazovaniya	Ministry of Higher Education of the U.S.S.R.
NIIOSP	Nauchno-Issledovatel'skii Institut Osnovanii i Podzemnykh Sooruzhenii	Scientific Research Institute for Foundations and Underground Structures
NIIVT	Novosibirskii Institut Inzhenerov Vodnogo Transporta	Novosibirsk Institute of Water Transportation Engineers

Abbreviation	Full name (transliterated)	Translation
NIIZhB ASiA SSSR	Nauchno-Issledovatel'skii Institut Betona i Zhelezo-betona Akademii Stroitel'-stva i Arkhitektury SSSR	Scientific Research Institute of Concrete and Reinforced Concrete of the Academy of Construction and Archi-tecture of the U.S.S.R.
NIMI	Novocherkasskii Inzhenerno-meliorativnyi Institut	Novocherkassk Institute of Reclamation Engineering
NIS Gidroproekta	Nauchno-Issledovatel'skii Sektor Vsesoyuznogo Pro-ektno-Izyskatel'skogo i Nauchno-Issledovatel'skogo Instituta imeni S. Ya. Zhuk	Scientific Research Depart-ment of the All-Union De-sign and Scientific Research Institute for Hydro Structures imeni S. Ya. Zhuk
NIS Orgenergostroi	Nauchno-Issledovatel'skii Institute po Proektiro-vaniya i Organizatsii Energeticheskogo Stroitel'-stvo	Scientific Research Institute for the Design and Planning of Power-engineering Construction
NITO	Nauchnoe Inzhenerno-Tekh-nicheskoe Obshchestvo	Scientific Engineering and Technical Society
Orgres	Gosudarstvennyi Trest po Organizatsii i Ratsionali-zatsii Elektrostantsii	State Trust for the Organi-zation and Efficient Plan-ning of Electric Power Plants
Raznoeksport	Vsesoyuznoe Eksportnoe Ob'-edinenie (Ministerstva Vneshnei Torgovli)	All-Union Association of Export Agencies of the Ministry of Foreign Trade of the U.S.S.R.
SANIIRI	Sredneaziatskii Nauchno-Iss-ledovatel'skii Institut Irrigatsii	Central Asia Scientific Re-search Institute for Irri-gation
SAOGIDEP	Sredneaziatskoe Otdelenie Gidroenergoproekta	Central Asia Branch of Hydro Power Plants and Develop-ments
SNiP	Stroitel'nye Normy i Pravila	Construction Standards and Rules
Soyuzdornii	Gosudarstvennyi Vsesoyuznyi Dorozhnyi Nauchno-Issledo-vatel'skii Institut	State All-Union Scientific Institute for Road Construc-tion

Abbreviation	Full name (transliterated)	Translation
Stalingradgidrostroi	Upravlenie po Stroitel'stvu Stalingradskogo Gidrouzla	Main Administration for the Construction of the Stalingrad Hydroelectric Development
Tashenergo	Tashkentskoe Raionnoe Upravlenie Energeticheskogo Khozyaistva	Tashkent District Administration for Power Engineering and Power Construction
TNISGEI imeni A. V. Vintera	Tbilisskii Nauchno-Issledovatel'skii Institut Sooruzhenii i Gidroenergetiki imeni A. V. Vintera	Tbilisi Scientific Research Institute of Construction and Hydro Power Engineering imeni A. V. Vinter
Trest Gidromekhanizatsiya	Vsesoyuznii Trest po Gidromekhanizatsii	All-Union Trust for Hydro Mechanization
TsNIIS Mintranstroi	Tsentral'nyi Nauchno-Issledovatel'skii Institut Ministerstva Transportnogo Stroitel'stva	Central Scientific Research Institute of the Ministry for Transportation Construction
TUiT	Tekhnicheskie Usloviya i Trebovaniya	Technical Specifications and Requirements
Ukrgidep	Ukrainskii Gosudarstvennyi Proektnyi Institut	Ukrainian State Design Institute
UkrVODGEO	Nauchno-Issledovatel'skii Institut Vodosnabzheniya, Kanalizatsii, Gidrotekhnicheskikh Sooruzhenii i Inzhenernoi Gidrologii Akademii Stroitel'stva i Arkhitektury Ukr. SSR	Scientific Research Institute of Water Supply, Canalization, Hydro Structures and Engineering Hydrology, of the Academy of Construction and Architecture of the Ukrainian SSR
Uzbekenergostroi	Uzbekskii Trest Energeticheskogo Stroitel'stva	Uzbek Trust for Power Engineering Construction
VASKhNIL	Vsesoyuznaya Ordena Lenina Akademiya Sel'sko-Khozyaistvennykh Nauk imeni V.I. Lenina	All-Union Academy of Agriculture imeni V.I. Lenin
VNII	Vsesoyuznyi Nauchno-Issledovatel'skii Institut	All-Union Scientific Research Institute

Abbreviation	Full name (transliterated)	Translation
VNIIG imeni B.E. Vedeneeva	Vsesoyuznyi Nauchno-Issledovatel'skii Institut Gidrotekhniki imeni B.E. Vedeneeva	All-Union Scientific Research Institute of Hydraulic Engineering imeni B.E. Vedeneev
VNIIGS	Vsesoyuznyi Nauchno-Issledovatel'skii Institut Gidrotekhnicheskikh i Sanitarno-Tekhnicheskikh Rabot	All-Union Scientific Research Institute of Hydro- and Sanitary Engineering
VODGEO	Vsesoyuznyi Nauchno-Issledovatel'skii Institut Vodosnabzheniya, Kanalizatsii, Gidrotekhnicheskikh Sooruzhenii i Inzhenernoi Gidrologii	All-Union Scientific Research Institute for Water Supply, Sewer Systems, Hydrotechnical Structures and Engineering Hydrogeology
YuzhNIIGiM	Yuzhnyi Nauchno-Issledovatel'skii Institut Gidrotekhniki i Melioratsii	Southern Scientific Research Institute of Hydraulic Engineering and Reclamation
Zakmetallurgstroi	Zakavkazkoe Upravlenie po Stroitel'stvu Metallurgicheskikh Predpriyatii	Transcaucasian Main Administration for the Construction of Metallurgical Works

Foreword

In 1958 the VNIIG imeni B. E. Vedeneev published a collection of abstracts of scientific research works carried out at the Institute during the year of 1957.

Shortly after publication of this first collection VNIIG approached a series of scientific research and educational institutes as well as certain design and building agencies, laboratories, etc., with the request to participate in the publication of a common collection of abstracts of scientific studies carried out during the year of 1958.

Such collections are useful, not only because they facilitate the coordination of scientific work of the participating institutes, but also because they make available to a large circle of readers the results [even in a somewhat digested form] of scientific investigations.

The collection of abstracts is divided into the following parts:

I Design of hydro structures for hydro and thermal power plants
II Stability of hydro structures and their foundations
III Building materials
IV Hydraulics
V Hydro construction and its organization
VI Operation of hydroelectric power plants and of their equipment
VII Investigations on hydro turbine units and other hydraulic machines
VIII Hydrotechnical structures of thermal power plants
IX Summary of advanced methods and application of scientific findings

Twenty-three various scientific agencies listed below participated in the publication of this collection.

The Institute intends to publish yearly such collections of abstracts of studies carried out in the preceding year and kindly requests all the agencies that took part in the preparation of this collection to write about their impressions and to advance suggestions for further improvement.

The collection has been prepared for publication by the Department of Standards and Engineering Information of the Institute under the guidance of I. A. Girshkan, Senior Research Worker, Candidate of Technical Sciences.

List of institutes and the number of studies carried out by each of them in each of the nine parts

No.	Name of agency	Parts									
		I	II	III	IV	V	VI	VII	VIII	IX	Total
1	AzNIIGiM	-	-	-	3	-	-	-	-	-	3
2	BPI	-	1	-	-	-	-	-	-	-	1
3	ENIIZiM	-	-	-	1	-	-	-	-	-	1
4	GIRDORYBPROEKT . . .	-	-	-	1	-	-	-	-	-	1
5	GISI imeni V. P. Chkalova	-	-	-	-	1	-	-	-	-	1
6	IEVKh AN KIRG. S. S. R. . .	1	1	-	3	-	-	-	-	-	5
7	IGiG AN Ukr. S. S. R. . . .	2	1	-	9	1	-	-	-	-	13
8	InEG AN ARM. S. S. R. . . .	-	-	-	8	-	-	-	-	-	8
9	KIIVKh	1	-	-	1	-	-	-	-	-	2
10	LGMI	-	-	-	1	-	-	-	-	-	1
11	MIIVKh imeni V.R. Vil'yamsa	1	3	-	1	-	-	-	-	1	6
12	MISI imeni V. V. Kuibysheva	3	-	-	5	3	-	-	1	-	12
13	NIIVT	-	-	-	1	-	-	-	-	-	1
14	NIIZhB ASiA S. S. S. R. . .	3	-	1	-	-	-	-	-	-	4
15	NIMI	-	-	-	9	-	-	-	1	-	10
16	NIS Gidroproekta	5	14	4	16	-	3	5	-	-	47
17	NIS ORGENERGOSTROI . .	-	-	-	-	8	-	-	-	-	8
18	TNISGEI imeni A. V. Vintera	4	5	-	1	-	1	1	-	-	12
19	UkrVODGEO	-	-	-	2	-	-	-	1	-	3
20	VNIIG imeni B. E. Vedeneeva	29	13	17	42	10	9	4	13	12	149
21	VNIIGS	1	-	-	-	-	-	-	-	-	1
22	VODGEO	1	-	-	3	-	-	-	-	-	4
23	YuzhNIIGiM	1	-	-	4	-	-	-	-	-	5
	Total	52	38	22	111	23	13	10	16	13	298

PART I

DESIGN OF HYDRO STRUCTURES FOR HYDRO AND THERMAL POWER PLANTS

VNIIG IMENI B. E. VEDENEEV DEPARTMENT FOR HYDRO STRUCTURES

Head: Professor A. Z. Basevich

LABORATORY OF ENGINEERING STRUCTURES

Head: K. A. Mal'tsov, Candidate of Technical Sciences, Senior Research Worker

INVESTIGATION OF THERMAL STRESSES IN THE DAM OF THE SANMENHSIA HEP *

Responsible for Research: P. I. Vasil'ev, Candidate of Technical Sciences, Senior Research Worker

The purpose of this study has been to select the suitable heat conditions and a system of structural division of the dam, which would prevent the appearance of large cracks in the structure.

The analysis of climatic factors and characteristic features of the structure permits conclusions to be drawn on the expediency of dividing the dam into vertical sections with application of artificial cooling of the concrete masonry. In the internal areas of the dam the longitudinal blocks may be divided in sections. In blocks of considerable length the temperature of the concrete masonry should be kept below 32° C, therefore the concrete mix should be placed at temperatures not above 17° C, the cement content should be 160kg/m^3, and the heat liberation in the concrete mix 60-70 kcal/kg.

The temperature at which longitudinal joints would seal up should be close to the mean annual temperature of the air, i. e. approximately +15° C. This can be attained through artificially cooling the concrete masonry by water passing through pipes.

The temperature of the concrete masonry can be lowered by cooling the mixing water and possibly also the coarse aggregates. The distance between the joints can be taken as equal to the length of a block, i. e. approximately 22-23 m.

* [On the Hwang Ho River in China.]

DEVELOPMENT OF IMPROVED METHODS FOR CALCULATION OF CONCRETE HYDRO STRUCTURES BY TAKING INTO CONSIDERATION THERMAL STRESSES AND CREEP OF CONCRETE

Research Team: P. I. Vasil'ev, Candidate of Technical Sciences, Senior Research Worker
Professor A. V. Belov, Doctor of Technical Sciences

This work is a continuation of investigations conducted for a number of years in the laboratory of engineering structures. It deals with practical methods for the calculation of thermal stresses in concrete masonry and with the evaluation of the effect of the hardening temperature on stresses in concrete. It also includes suggestions for the design of plain and reinforced-concrete structures with allowance for thermal stresses.

From the investigation conducted in 1957-58 the following conclusions can be drawn:

I. The hardening temperature has a considerable influence on the rate of heat liberation, increase in strength and hardness of concrete.

For practical calculations, however, heat liberation, as well as changes in the modulus [of elasticity] and strength of concrete can be considered as a function of time.

The study proved that, assuming

$$\frac{d\varepsilon}{dt} = F_1(T_\varepsilon,\ \varepsilon);\quad \frac{dE}{dt} = F_2(T, E);\quad \frac{dR}{dt} = F_3(T, R),$$

where $\frac{d\varepsilon}{dt}$ = rate of heat liberation at a given point as a function of hardening temperature (T_ε) and time;

$\frac{dE}{dt}$ = increase in hardness of concrete at a given point as a function of hardening temperature and time;

$\frac{dR}{dt}$ = increase in strength at a given point as a function of hardening temperature and time;

and knowing the curves of heat release and of the increase of the modulus under isothermic conditions, a numeric solution to the problem of thermal stresses in blocks of the embedded or freely supported slab type can be obtained. Thus, the problem of determining thermal stresses in elements, for which the hypothesis of direct normals holds good, can also be considered as almost solved when the effect of the hardening temperature on the heat liberation and on the increase of modulus of elasticity of concrete are taken into account.

II. The thermal interaction between concrete blocks can be determined numerically by the method of finite differences. The results of calculations show that, when a new block is more or less quickly placed on a previously cast one, the thermal interaction is of a short duration, lasting from three to ten days after concreting. Afterward, the temperatures of the adjoining blocks are leveled out and the blocks undergo cooling as a solid massif. This permits the value of the mean temperature throughout the width of the blocks to be introduced into the calculation

The study of the interaction of forces between blocks is in general difficult, still more so because of the varying stiffness of concrete during its increase in strength. The problem of determining the thermal stresses in

a dam section, starting to cool at one of its surfaces, consists actually in determining the effect of load applied on the surfaces of the joints, the load distribution depending on the temperature field.

This problem can be solved for certain cases by using the lines of influence of stresses.

The study presents practical suggestions for the determination of temperatures and thermal stresses in massive concrete.

In case of a one-dimensional temperature distribution, the temperatures can be determined by the graphic method developed by Professor Schmidt. In structures where the temperature distribution is of a two-dimensional nature, the temperatures may be determined by the method of finite differences. For three-dimensional temperature distribution, the temperatures will be determined by the method of finite differences, the calculations being done in a tabular form, whereas for similar permanent boundary conditions on all the six faces of the hexagonal massive concrete, the calculation results will be presented as a parallelepiped – by the method of multiplication of three-linear temperature distributions.

Thermal stresses in structures of the slab type with nonrestricted thermal deformation can be determined by using the hypothesis of direct normals applied to the median area of the slab according to the formula

$$\sigma = \frac{E\alpha}{1-\nu}[T_{av} + 2B_x - T_x],$$

where T_{av} = the mean temperature throughout the thickness of the slab at the time of investigation, and equals

$$T_{av} = \frac{\int_{-l}^{l} T_x d_x}{2l}.$$

B is defined by

$$B = \frac{3S}{4l^3},$$

where S = the static moment of the area delimited by the temperature curve at the given moment and by the curve of the initial distribution of temperatures in relation to the axis of the slab, and equals

$$S = \int_{-l}^{l} T_x x d_x,$$

where l = half the thickness of the dam.

In the case of bilateral cooling of a freely supported slab or in the particular case of symmetric cooling of the slab at a constant temperature, as well as in a few other cases, formulas are given for the determination of σ of the surroundings.

The study gives indications as to the determination of thermal stresses in blocks embedded at the bases and in those laid on vertical sections in case of a one-dimensional temperature distribution.

It is also explained how to include the creep of concrete into the calculation of thermal stresses.

The study finally gives instructions on the calculation of the system of artificial cooling of concrete masonry.

LIGHTWEIGHT (BUTTRESS) TYPES OF HIGH CONCRETE DAMS BUILT ON WATER-RICH RIVERS OF THE U.S.S.R.

Responsible for Research: A. V. Shvetsov, Candidate of Technical Sciences

This study, published in 1958, is a continuation of the study on the same subject published in 1957.

The paper contains a further analysis of profiles of lightweight types of dams suggested previously, i. e. of dams with wider joints and also of dams erected by the sectional method, having a prestressed upstream face. Particular attention is paid to the combined use of certain structural methods allowing the reduction in weight of the dam, and data are presented on the efficiency of these methods.

The results of these investigations form a basis for further laboratory research.

One of the above lightweight types of dams (a dam with wide joints and longitudinal galleries) and its calculation method have been already realized.

MODEL TESTS OF THE KRASNOYARSK HEP DAM

Responsible for Research: K. A. Mal'tsov, Candidate of Technical Sciences, Senior Research Worker

Research team: A. M. Arkhipov, Senior Engineer
I. B. Sokolov, Senior Engineer
A.A. Khrapkov, Engineer

Until recently, the study of stresses in hydro structures has been conducted at the VNIIG on small-scale models made of elastic materials, while other countries are widely using the method of bench-testing on models made of brittle materials.

In 1958, in connection with the necessity to investigate the Krasnoyarsk HEP dam, the laboratory decided to adopt bench-testing of brittle models. The research has been conducted in the following directions:

1. A special test bench has been designed and constructed for investigations of gravity dams (including the Krasnoyarsk dam) on models made to a scale of 1:40 to 1:50.

The bench, made of reinforced concrete having the following dimensions: length 11m, width 4.6 m and height 7.3m, permits the simulation of loads and forces acting on the dam and on its foundation, such as hydrostatic pressure at the upstream face, the weight of the water column acting on the base of the dam, dead weight of the dam and its foundations, water pressure acting on the foundation from the headwater and tailwater side.

The bench was designed for testing the rock-foundation models. The height of the foundation was equal to three fourths of the dam height.

The testing time can be reduced by using two models. While one is tested, the other is prepared for tests.

Models of five sections of the Krasnoyarsk dam will be tested on the bench so as to simulate conditions representing the actual two-dimensional stresses in the foundation of the dam. The model of the central section will be of exactly the same geometric form as the original, reduced to a scale of 1:50, while the four other models are erected to a reduced height The stresses in these four models are not being investigated, since they act on the foundation with the same intensity as the central section which is being thoroughly investigated.

2. Appliances were designed and constructed permitting the creation of stresses in these models.

Hydrostatic pressure at the upstream face of the dam is created by means of jacks which produce a load on the seven areas according to the law of hydrostatic pressure. The system permits simulation of gradual filling of the reservoir up to the rated level, and gradual pressure increase on the dam model up to its destruction.

To simulate the dead weight of the dam on the model, an arrangement of metal levers and ties has been provided.

The uniform distribution of loads on the model, simulating the water pressure on the dam foundation from the upstream side, can be attained by using rubber cylinders (bottles).

Horizontal forces acting on the loading sections are created by means of jacks whose load is applied exactly to the center of gravity of the graph of hydrostatic pressure. Vertical forces on the loading sections are produced by jacks in accordance with the law of distribution of weight.

3. Special material of given modulus of elasticity and strength has been selected for models of the dam and the rock foundation. Gypsum and cement are used as binders and coal slag as filler.

4. A method of stress measurement in models of dams and of their foundations has been developed and special apparatus were designed for measuring stresses at 350 check points.

DEVELOPMENT OF METHODS FOR INVESTIGATIONS ON MODELS
MADE OF BRITTLE MATERIALS

Responsible for Research: K. A. Mal'tsov, Candidate of Technical Sciences, Senior Research Worker

Research team: A. M. Arkhipov, Senior Engineer
I. B. Sokolov, Senior Engineer
S. S. Antonov, Junior Research Worker
A. A. Khrapkov, Engineer

Calculation data for the design, as well as data on the actual safety factors of hydro structures, can be experimentally checked on test models made of brittle material.

Until recently no sufficient attention has been paid in the U. S. S. R. to the problem of tests on models made of brittle materials. The first task put before the laboratory for engineering structures concerning the problem of model testing was that of designing the required test benches.

The following work was done in 1958:

1. A bench was designed and constructed for tests on models of gravity dams under conditions of the two-dimensional problem.

2. A special arch-type bench was designed and partly constructed for testing models of arch- and dome-type dams under conditions of the three-dimensional problem.

The dimensions of the benches are so chosen as to permit the modeling of dams up to 150 m high. The scale for the gravity-type bench is 1:40 to 1:50, for the arch-type bench from 1:40 to 1:100. In addition, both types of benches permit the modeling of rock foundations to the same scale as the dam.

3. Loading devices were designed and constructed, capable of simulating, on models, the complex forces acting on the structure. The intensity of loads can be increased on these devices up to destruction of the models.

The model-loading devices are provided with appliances permitting the application of load on the upstream face of the model subjected to horizontal forces distributed according to the triangular or trapezoidal pressure diagram on the models subjected to three-dimensional forces of the dead weight with or without taking into account the weight of the water column acting on the dam; on the models of rock foundation with or without taking into account the headwater and tailwater pressure, as well as with or without taking into account the three-dimensional forces of the dead weight of the upper layer of rock foundation; on the arch-type models so as to simulate the effect of the temperature on the structure.

4. The proper materials have been selected for test models, prepared of gypsum and cement binders, and slag fillers.

It has been proved that, by using slag fillers, it is possible to select materials which will possess the necessary elasticity and structural characteristics.

5. A research method has been developed and apparatus constructed for measuring the state of stress of dam models.

Responsible for Research: S. S. Antonov, Junior Research Worker

The aim of this study has been to investigate the new design of a three-hinged (three-arch) dam developed in 1957 by V. I. Kravtsov.

The study of operation of a three-hinged arch dam proceeded in several stages:

a) development of a calculation method;

b) a preliminary design of structures for a comparative evaluation of the efficiency of the new design;

c) design of a three-hinged arch for the Ladzhanura HEP dam;

d) tests of hinges on 1:10 and 1:4 scale models.

The investigations gave the following results:

1. Three-hinged arch dams are the most suitable when a conventional arch dam, designed to be embedded into the foundation and into the banks, possesses mean central angles $\varphi < 100°$ at $\frac{r}{t} < 10$, and an allowable compressive strength $[R]$ of 50 kg/cm^2.

2. This design of a dam is particularly suitable for regions where the temperature variations are within the limits of $\Delta t \pm 20°C$.

3. No axial tensile stresses, resulting from basic loads such as hydrostatic pressure nonuniform temperature, settling of abutments, appear in three-hinged arch dams; moreover, this type of dam satisfactorily resists additional stresses caused by pressure of interstitial (pore) water, seismic effects, etc.

4. It has been suggested to design the hinges for a three-hinged arch as sections reduced to about 60 % of the arch thickness.

5. The allowable compressive stresses in the arch elements should be lower than half the strength of an equivalent concrete prism.

The results of the report can be used both in calculation and design as has already been done by Gidroproekt for the design of the Ladzhanura dam.

Work should be continued on the study of the interaction between adjoining arch elements and of a suitable design for horizontal joints and their fillers. This work should be done both under field conditions (on an experimental dam) and in a laboratory (tests of model joints).

INVESTIGATIONS OF THE CONSOLIDATION OF PIERS OF THE POWERHOUSE OF THE DNEPROGES IMENI V. I. LENIN

Responsible for Research: Professor A. Z. Basevich

Research Team: A. M. Arkhipov, Senior Engineer
M. M. Korolev, Junior Research Worker

Laboratory research of this problem has been required by the necessity to repair a number of piers with considerable cracks, caused by explosions during World War II.

As a result of investigation, it has been suggested to consolidate the piers by:

strengthening the superstructure (above water) of the piers with vertical steel anchors;

providing, apart from these ties, inclined steel ties on those piers most likely to be damaged to prevent their destruction during repair of their submerged sections.

LABORATORY FOR FIELD RESEARCH

Head: V. V. Blinkov, Candidate of Technical Sciences, Senior Research Worker

INVESTIGATION OF HYDRO STRUCTURES OF THE KAMA HEP

Responsible for Research: M. B. Ginzburg, Candidate of Technical Sciences, Senior Research Worker

The investigations were carried out between 15 October 1955 and the end of 1958. The following problems were studied:

1) settlement of concrete structures and deformations of their joints;
2) uplift in the foundation of concrete structures, and the efficiency of the drainage systems;
3) behavior of the grout curtain;
4) seepage through concrete and earth structures;
5) chemical processes developing in the foundation of the structure;
6) strength of concrete structures;
7) conditions of the embankments of earth dams;
8) vibration of structures;
9) general problems of operation of hydroelectric power plants;
10) methods for the prevention of head losses;
11) state of the hydraulic and mechanical equipment.

The investigation carried out in 1958 indicated the following:

1. Settlement of concrete structures ceased almost completely; variation in elevation marks is due to temperature variations in the concrete;

2. Variation in width of joints has a cyclic nature, also determined by temperature variations in the concrete;

3. The uplift in the foundation of the spillway HEP is small; the principal drop of uplift pressure occurs at the grout curtain and the fore apron; below the structure, the piezometric pressure equals almost the piezometric pressure at the tailwater side; the general pattern of uplift pressure can

be seen from lines of equal piezometric pressure at the plant site (see Figure 1).

4. The grout curtains in a number of sectors are of poor quality; high specific water absorption has been noticed in the control well - in some areas over one 1/min per running meter; the specific water absorption increases simultaneously with increases in the discharge through piezometric wells; the salinity of the ground waters decreases; all these signs prove that piping phenomena noticed are due to chemical processes; it has been suggested to inject without any delay additional cement grout of improved quality (increased working pressure, careful check of cement quality, use of clay admixtures, vibration-grinding of cement, etc.).

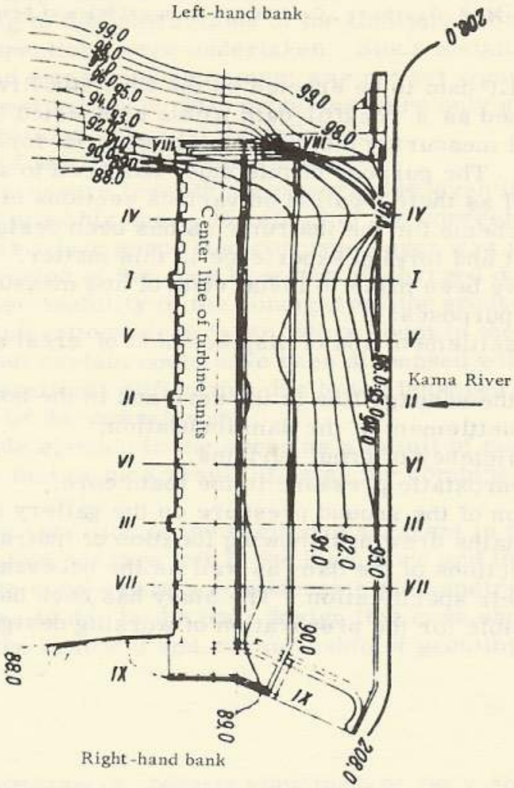


FIGURE 1. Lines of equal piezometric pressure of ground waters at the base of the Kama HEP as taken on 8 December 1958

5. Since these investigations did not permit an evaluation of the total seepage discharge, it was intended to carry out in 1959 a special research on electrohydrodynamic analogy models.

6. The work on the erection of earth structures initiated in 1958 is not yet completed; it is necessary to complete all earthwork during 1959.

7. Operational shortcomings have been found, caused by faulty selection of type and design of structure and, where it was possible, suggestions for their elimination have been made.

8. The vibrations in structures are low; they do not exceed the values for HEPs with separate (not built-in) powerhouses.

Careful attention should be paid in future to the study of seepage and chemical processes in the foundation of structures.

CONTROL AND MEASURING INSTRUMENTS FOR THE CHARVAK HEP DAM

Responsible for Research: M. B. Ginzburg, Candidate of Technical Sciences, Senior Research Worker

The Charvak HEP dam to be erected on the Chirchik River is 150 m high. It has been designed as a rockfill dam with a compacted loam core. The use of control and measuring instruments is essential for normal operation of the structures. The purpose of this study has been to select suitable instruments, as well as their location on various sections of the Charvak dam.

The location scheme for the instruments has been designed after careful study of Soviet and foreign experience in this matter.

Provisions have been made to instal control and measuring instruments for the following purposes:

a) control of settlement and of displacement of crest and embankments of the dam;

b) control of the seepage line in the core and in the body of the dam;

c) control of settlement of the dam foundation;

d) study of efficiency of grout curtains.

e) study of hydrostatic pressure in the loam core;

f) investigation of the ground pressure on the gallery ceiling.

The study contains drawings showing location of instruments in plan and throughout the sections of the dam, as well as the necessary number of instruments and their specification. The study has been handed over to the institute responsible for the preparation of working designs.

INVESTIGATION OF SEEPAGE, UPLIFT PRESSURE AND GENERAL CONDITION OF HYDRO STRUCTURES OF THE IRKUTSK HEP ON THE ANGARA RIVER

Responsible for Research: M. B. Ginzburg, Candidate of Technical Sciences, Senior Research Worker

The investigation program included a study of: settlement of earth and concrete structures; deformation of joints; seepage through foundations of concrete structures; seepage through earth dams; uplift pressure within the foundation of the powerhouse; scouring of the tailwater channel.

Fluctuations in the headwater and tailwater level, variations in the water and air temperature in the storage reservoir, as well as discharge rate of the Angara River, were also investigated.

A systematic inspection of structures started in April 1958 and was carried out according to the instructions of the Institute. Until April 1958 only sporadic inspections were undertaken. Since installation of the required control and measuring equipment was not yet completed at the beginning of the investigation, certain problems were only superficially treated.

These investigations indicated the following:

1) cessation of settlement of the structures;

2) low uplift pressure beneath the powerhouse, even in front of the grout curtain. This is probably due to the effect of the concrete layer over the rock which acts as a fore apron (the concrete cover was placed as a protection against erosion of the rock by water discharged during construction work and to the permeability of the concrete and the grout curtain. It seems that if these considerations were taken into account in the design of the dam, the expensive grout curtain could have been dispensed with;

3) certain operational difficulties due to the leakages from bottom outlets (sluiceways) of the powerhouse;

4) considerable erosion in the apron as a result of concentrated action of discharge jets, that is, as a result of using the combined type of power plant;

5) satisfactory condition of the earth dams, except in the area of the temporary navigation passage left in the loam core and the cement-grouted rock-fill under the core, where a considerable piezometric head was recorded. This pressure is due to the poor design (the core should have been carried down to the bedrock) and workmanship of grouting operations.

INSPECTION OF CONCRETE STRUCTURES OF THE VARZOB CASCADE OF POWER PLANTS

Responsible for Research: M. B. Ginzburg, Candidate of Technical Sciences, Senior Research Worker

The purpose of the inspection has been a detailed, mainly visual, examination of all concrete structures of the cascade system and an analysis of the operational data.

The inspection was carried out in September and October 1958. In addition to the thorough examination of the concrete structures, all available maintenance reports and especially the surveying logs were carefully

studied. Record cards for every structure were consulted before the examination. In general, all hydro structures of the cascade system have been found in a satisfactory condition. A detailed description of the structures and their actual state of repair as revealed by inspection are given in the paper. Attention was drawn to the following necessary measures:

a) reinforcement of the right-hand banks of the Varzob River in the vicinity of the headworks;

b) cleaning of the diversion channel and a check-up of the Kharangonskii water main as soon as shutting down of the cascade will be possible;

c) replacement of the wooden conduit in the head section by a steel conduit and installation of a settlement-compensating device;

d) repair of all leakages in the area of the forebay of the power station No. 2, whose foundation contains interlayers of loesslike loam deposits.

The report was passed on to the Tadzhikenergogaz for further use.

FIELD INVESTIGATIONS OF THE STATIC BEHAVIOR OF HYDRO STRUCTURES AT THE KAKHOVKA HYDRO DEVELOPMENT AND OF THE HYDRODYNAMIC ACTION OF WATER CURRENT ON THESE STRUCTURES

Responsible for Research: S. Ya. Eidel'man, Candidate of Technical Sciences, Senior Research Worker

In 1958 a study was carried out of structures at the Kakhovka hydro development with the help of existing control and measuring equipment.

Settling of the structures was advancing, though not significantly. For example, the average rate of settling of separate dam sections, during eleven months of 1958, did not exceed 8 mm, the full settlement varying between 144 and 221 mm. The respective rate for the powerhouse was 4 mm and 158-185 mm; for the lock – 8 mm and 202-240 mm; and for the pier – 30 mm and 369-1153 mm.

Horizontal displacement of the dam was checked by the "alignment" method using check points located on the dam crest. These displacements were found to have a seasonal character and to vary with the water and air temperature. In summer the dam crest shifts up to 20 mm in the headwater direction, while in winter the movement is directed toward the tailwater section, the displacement not exceeding 10 mm.

Investigations of the uplift pressure within the dam foundation were carried out before 1958, and indicated the satisfactory state of seepage-preventing structures. About 85% of the existing uplift pressure is dissipated by the upstream apron (and apparently also by the underlying deposits of "linear" estuary silts), while 10-12 % of the uplift pressure is taken up by sheet piling.

Following the 1958 flood, the uplift pressure at the dam base increased somewhat. The piezometric level at the section between the upstream and downstream sheet-piling cutoffs exceeded the tailwater level by about one meter, a fact which is probably due to the partial clogging of the drains. The same pattern can be observed in all three sections where piezometers were installed. In the powerhouse of the HEP a departure from the norm

was noted at one point only, the upstream piezometer in one of the sections indicated a level approaching that of the headwater level. This is probably due to possible damage to the fore apron near the area of the piezometer.

Inspection of strain-gage operation was carried out. In the course of six years about 50% of the gages became inoperable. Readings taken in 1958 of underground dynamometers indicated relative stability. Contact stresses rarely exceeded 5-6 kg/cm^2 (not taking into account uplift pressure) and reached (only in some places) 9 kg/cm^2. These stresses are well within the safety limits. Strain-gage measurements in reinforcement bars also showed stable values of stresses. Tensile stresses larger than 300 to 400 kg/cm^2 were not recorded except in a few single cases.

INSPECTION OF THE WOODEN PENSTOCK OF THE NIVA HEP

Responsible for Research: S. Ya. Eidel'man, Candidate of Technical Sciences, Senior Research Worker

In 1957 investigations were carried out of the wooden penstock of the Niva HEP No. 2 in order to determine future operations.

In 1958 sample staves were tested and a conclusive report about the condition of the penstock was issued.

The penstock, 4 m in diameter and 100 m long, was put into operation in 1934 and worked uninterruptedly until the outbreak of the war. In 1941 the penstock was damaged. Having been repaired, it has operated satisfactorily until the present. The general condition of the outside of the penstock is basically satisfactory. Inspection of the inside revealed a number of various defects in the staves, the most serious defect being exfoliation of the wood. In 1955 Lengidep worked out a project for the restoration of the penstock and full replacement of the staves. The design was based on an estimated 20-year lifespan of the pipes.

Inspection of the penstock in 1957 and the testing of sample staves was done with the participation of the Leningrad Forestry Academy imeni Kirov. The examination showed that the area most damaged was the lower section of penstock No. 2, and replacement of staves in that section was therefore recommended.

The three remaining penstocks are in satisfactory condition, neither immediate restoration nor general overhauling being required; however the replacement of damaged staves was recommended. Considering the deterioration of physical and mechanical properties of the staves, special measures for maintenance and repair of the penstocks have been recommended.

DRAFT OF INSTRUCTIONS FOR THE INSTALLATION AND MAINTENANCE OF CONTROL AND MEASURING EQUIPMENT FOR HYDRO STRUCTURES, AND FOR THE PRIMARY PROCESSING OF DATA (FIRST DRAFT)

Responsible for Research: S. Ya. Eidel'man, Candidate of Technical Sciences, Senior Research Worker

Reliable operation and long service life of measuring instruments and equipment for structures depend, apart from good design and production

quality, upon their correct installation and building-in. Incorrect building-in and installation may damage the devices and affect the reliability of their readings. It is therefore necessary to prepare specifications and instructions for the personnel in charge of the building-in of instruments in concrete masonry. The first draft of these instructions deals with the installation, in hydro structures, of the following instruments: piezometers, underground dynamometers, piezodynamometers, strain gages for measuring stresses in reinforcements, remote-indication thermometers, remote-indication strain gages, devices for remote reading of width of cracks, and instruments for direct measuring of the stresses in concrete.

The design of instruments and their layout principles are not considered in this first draft. The instructions contain indications not only for the building-in of instruments, but also for the connection of the instruments to the measuring station.

A special chapter of the instructions deals with the determination of physical and mechanical properties of concrete and particularly of its modulus of elasticity and creep characteristics.

Two other chapters deal with the problems concerning the maintenance of built-in instruments and the primary processing of data.

VNIIG IM. VEDENEEV, LABORATORY FOR PHOTOELASTICITY

Head: Professor S. G. Gutman, Doctor of Technical Sciences

DEVELOPMENT OF METHODS FOR INVESTIGATING THE BEHAVIOR OF STRUCTURES FORMING A MONOLITHIC COMPLEX WITH THEIR FOUNDATION

Responsible for Research: Professor S. G. Gutman, Doctor of Technical Sciences

Research Team: N. S. Rozanov, Candidate of Technical Sciences, Senior Research Worker
N. S. Dement'eva, Senior Engineer
V. N. Mokrushin, Senior Engineer

The paper consists of four parts. Part I deals with complex theoretical-experimental methods for determining thermal or, respectively, uplift-pressure stresses within a hydro structure. The solution to this problem is divided into three consecutive stages.

1. At the first stage, the distribution of temperatures, or of uplift pressure, is determined, either by mathematical analysis or with the help of electrical-analogy models.

2. At the second stage, the particular solution to the problem is worked out directly from the potential field obtained in the first stage.

3. At the third stage, an auxiliary general solution is obtained which, when added to the particular solution arrived at earlier, gives the solution for zero boundary conditions. This auxiliary solution, which corresponds to a structural massif not influenced by temperature and uplift-pressure stresses, may be easily obtained with the help of a model, either by photoelastic or tensometric methods (stress-strain net).

The method is illustrated by analytical computations for a number of problems.

For an inhomogeneous complex formed by structures and their foundation, with different coefficients of heat conductivity, thermal expansion and seepage, the condition of continuity is ensured by introducing into the particular solution two additional functions of a complex potential. These functions correspond to the potential field of a uniform infinite plane with internal sources distributed over the rectilinear section of the interface (between both structural elements). These functions are determined either

1) analytically, with the help of the factor $\frac{1}{\pi} \ln \frac{z+a}{z-a}$, which undergoes at the interface a jump $\pm i$, or

2) on a two-dimensional electrical-analogy model.

Part II contains the theoretical basis of the method for determining residual stresses arising when one structural element is erected on another under load.

The theory of elasticity and of strength of materials assumes that an elastic body is not subjected to external forces as long as it is not yet in the finished state. In fact, external forces, e. g. the dead weight, one of the most important load factors in the design of massive structures, act from the very beginning of erection. The stressed state of a structure depends on the sequence in erection of the structure and the application of load. With a successive loading of the structure during its erection, initial stresses, which should be taken into account, are set up within the structure. The analytical method for the determination of residual stresses is based on the following three principal assumptions:

1. Addition of a new block, free of external load, to the basic block of a structure does not produce any additional stresses.

2. Upon relief of the load from the basic block, the adjacent additional block starts to work, resisting the restoring of the initial and deformed contour of the basic block. Interacting forces, such as normal stresses q and tangential stresses t, appear at the line of contact (interface) between the two blocks, which determine the over-all state of residual stresses in both blocks taken as a complex.

3. The distribution of the unknown stresses q and t is found from the condition of continuity of contact between both blocks.

This method made it possible to determine the diagrams of residual stresses resulting from the dead weight and hydrostatic pressure in [concrete] massifs composed of successively built-up vertical columns or horizontal blocks, or in massifs with triangular profile, composed of successively erected segments. The methods for the static computation of residual stresses are used for determining deformations which are due to shrinkage and heat of hydration in the concrete of the blocks being successively erected. With regard to experimental investigations, a general technique for testing on photoelastic models - the frozen-stress method - has also been developed.

Part III of the paper deals with the development of a method for investigating the behavior of structures and their foundation taken as a complex and assumed to operate under two-dimensional or three-dimensional stresses. The investigations have been carried out on plastic models by the photoelastic and tensometric methods. The latter, originally used only

for laboratory investigations of two-dimensional stresses, has later been adapted to three-dimensional problems as well. The method described has been applied to tests on a three-dimensional model of the upper head of a lock.

A considerable part of this chapter deals with the methods of preparing composite models for objects of heterogeneous materials, e. g. models for a structure-foundation complex.

The relationship between the stress-strain moduli of the building materials is used as a main criterion for simulating the heterogeneity of materials of these structures. The study describes various methods for simulating heterogeneous materials. Special attention is paid to problems of testing on models made of a glue and glycerine mixture and investigated by the tensometric method. According to this method, the properties of materials having different mechanical characteristics may be simulated on such models by using various compositions of the glue and glycerine mixtures. A method for selecting such mixtures having the required mechanical properties was set forth.

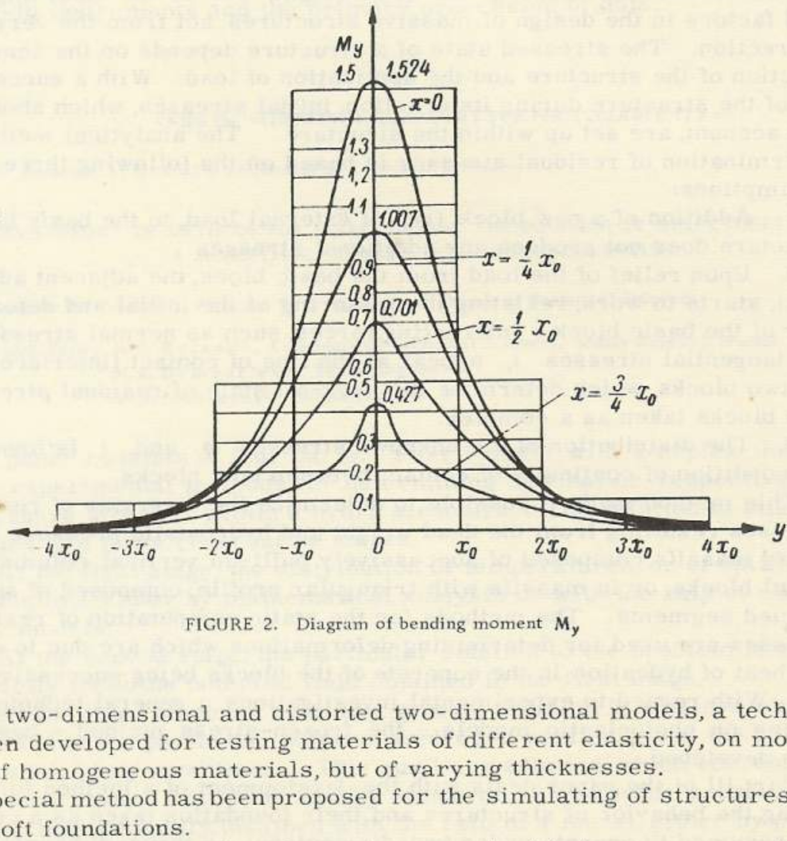


FIGURE 2. Diagram of bending moment M_y

For two-dimensional and distorted two-dimensional models, a technique has been developed for testing materials of different elasticity, on models made of homogeneous materials, but of varying thicknesses.

A special method has been proposed for the simulating of structures erected on soft foundations.

Results of model testing of structures made of heterogeneous materials are given in the paper.

The method may also be applied for investigation of structures made of the so-called "zonal" concrete, in which various sections are made of concrete of different grade, each corresponding to different stress-strain moduli. Investigation of such models showed that through the use of "zonal" concrete the state of stress in a structure may be controlled to a certain extent and stresses in a critical zone may be efficiently redistributed.

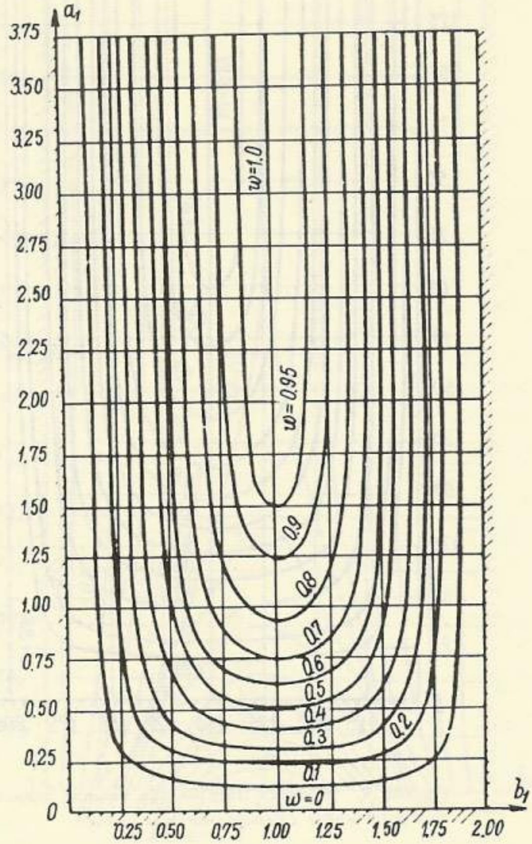


FIGURE 3. Influence lines for the bending stresses at a point where a bending force is applied

The state of stress of a model may be, under certain circumstances, affected by creep phenomena occurring in glue-glycerine composition mostly used for stress-testing models. Therefore, the laboratory investigated the plasticity properties of this composition by the tensometric method and by photographs of the deformations in the models. Special testing devices have been designed and constructed. As a result of the investigations, simulating of the creep behavior of concrete structures has been made possible.

Part IV deals with the theoretical solution to the problem of bending of a cantilever slab subjected to load on its free end.

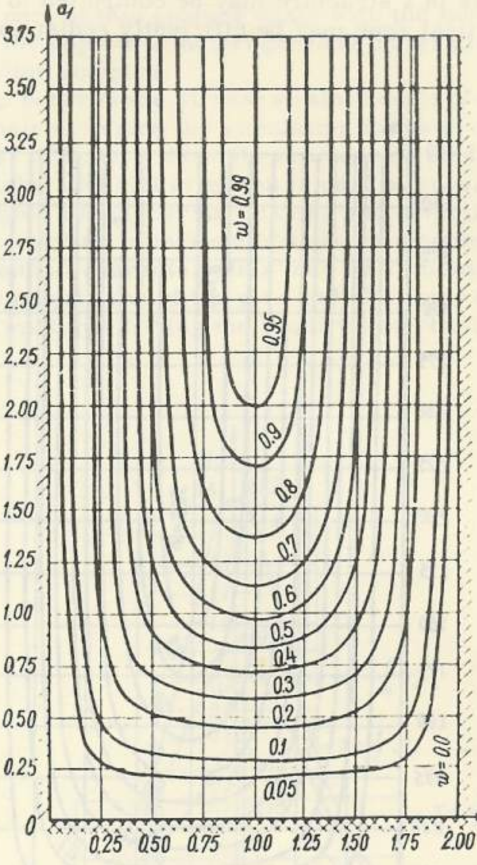


FIGURE 4. Influence lines for the bending stresses at a point where a bending force is applied

Section I gives a general solution to the problem for a slab considered as an infinite strip with one rigidly fixed end ($x = 0$) and the load applied at its free end ($x - x_0$). The relationship for deflection at the mid-span of the slab is taken as following:

$$w = [A\alpha x \operatorname{sh}\alpha x + B(\alpha x \operatorname{ch}\alpha x - \operatorname{sh}\alpha x)] \cos \alpha y +$$
$$+ [A_1 \alpha x \operatorname{sh}\alpha x + B_1(\alpha x \operatorname{ch}\alpha x - \operatorname{sh}\alpha x)] \sin \alpha y.$$

This solution is valid for the loads applied at the free end of a fixed-end slab in accordance with the law expressed by the Fourier series.

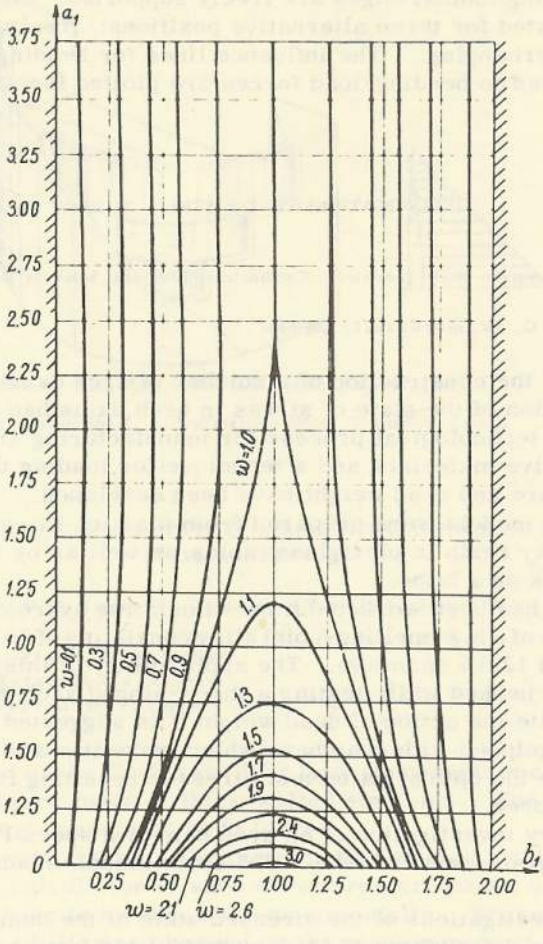


FIGURE 5. Influence lines for the bending stresses at a point where a bending force is applied

Section II deals with the problem of bending of a cantilever slab under a concentrated load applied at its free end. The function of deflection at the mid-span of the slab is given by

$$w = \int_{-\infty}^{\infty} \frac{q(\alpha y)}{2D\alpha^3} \frac{u(\alpha x_0)(\alpha x \operatorname{ch} \alpha x - \operatorname{sh} \alpha x) - v(\alpha x_0)\, \alpha x \operatorname{sh} \alpha x}{H} \cos \alpha y \, d\alpha.$$

Figure 2 represents the bending moments M_y, shearing force v_{xz} for different cross sections of a slab, and the support reaction Q_x of the fixed end of the slab. The diagrams of the bending moment M_y are also shown in Figure 2. Section 3 deals with the bending of a slab considered as a semi-infinite strip subjected to a single force acting at an arbitrary point of the slab. The longitudinal edges are freely supported; the transverse edges are investigated for three alternative positions: freely supported, rigidly fixed, and overhanging. The influence lines for bending stresses at a point subjected to bending-load forces are plotted (see Figures 3, 4, 5.).

INVESTIGATIONS OF THE STRESS OF ARCH DAMS

Responsible for Research: N. S. Rozanov, Candidate of Technical Sciences, Senior Research Worker

Research by: Ya. G. Skomorovskii, Engineer

In view of the construction of a number of arch dams in the Soviet Union, an investigation of the state of stress in arch dams has been undertaken.

In 1958 a technological process for manufacturing arch-dam models from optically active materials, and a technique for loading the models by hydrostatic pressure and dead weight, have been developed.

Arch-dam models were prepared from plastics by casting and polymerizing an ED-6 epoxy resin in plexiglass molds, as well as by mechanical machining suitable stock on a lathe.

A method has been worked out for simulating hydrostatic pressure on a model by use of a low-melting-point alloy consisting of 50 % bismuth, 25 % lead, 12.5% tin and 12.5% cadmium. The applicability of this alloy for loading the models was checked while testing a three-hinged arch dam.

To simulate the action of dead weight it is suggested to use a special centrifuge equipped with chambers which make it possible to heat the models and to freeze the optical pattern of stresses resulting from the action of the centrifugal force.

Laboratory investigation of an arch dam of a special design developed by VNIIG has been conducted on a 1:350 scale model, made of ED-6 epoxy resin.

Model investigations of the stressed state of the dam made it possible to determine the distribution of stresses resulting from hydrostatic pressure in different sections of the dam. The applicability of conventional computation methods for the investigations was also checked. The ratio between the moduli of elasticity of materials used for the foundation and the structures was $\frac{E_f}{E_{str}} = 1$. The model was loaded by means of the above alloy in accordance with the above technique (Figure 6).

On the basis of the model-test results, graphs of the stresses produced by the hydrostatic pressure on the dam have been plotted for horizontal sections of three arch elements. The pattern of stress distribution in the arches of the arch elements I, II and III shows the favorable stress distribution in

a three-hinged dam, the main acting forces being those of uniform compression (see Figure 7).

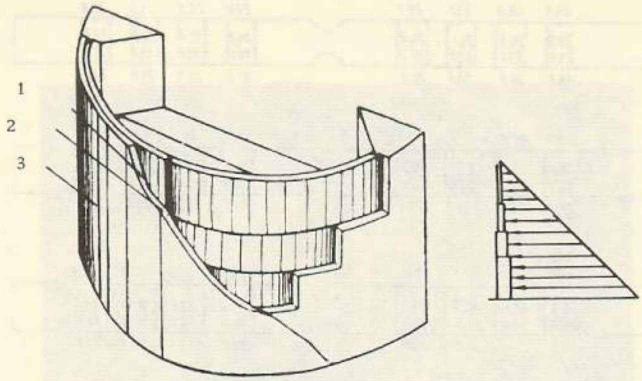


FIGURE 6. Arch-dam model made of ED-6 epoxy resin

1–thin rubber sheet; 2–low-melting alloy; 3–cardboard wall.

INVESTIGATION OF THE STATE OF STRESS IN THE UPPER HEAD OF THE PAVLOVSK HEP LOCK

Responsible for Research: N. S. Rozanov, Candidate of Technical Sciences, Senior Research Worker

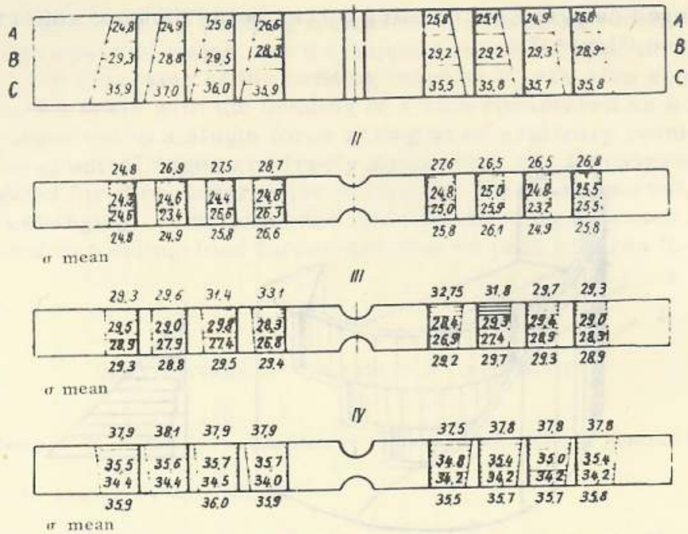
Research Team: A. K. Loginov, Senior Engineer
E. G. Tatarnikova, Engineer
V. N. Skorodumov, Engineer

The upper head of the Pavlovsk HEP lock represents a complex three-dimensional structure made of thick-walled elements. The structure is subjected to three-dimensional body forces.

In view of the complexity of the construction of lock heads, the analytical method for static calculation of such structures are based on numerous assumptions which have to be checked experimentally. Therefore, the laboratory of the OMIN VNIIG was charged by the management of the Pavlovsk HEP project with the experimental investigation of the state of stress of the lock head. Stresses caused by static loads such as hydrostatic pressure and dead weight of backfill behind the lock-gate recesses were investigated. Moreover, the thermal stresses in one of the sections of the lock wall were examined under conditions of a steady (hydraulic) regime.

Special methods have been developed by the OMIN laboratory for investigating the state of stress in the lock heads. Static stresses were determined by the tensometric method specially adapted to the solution of three-dimensional problems.

The laboratory also developed a technique for analyzing the thermal stresses. Investigation on test models (Figure 8) showed the structures to be subjected to a complex stress pattern.



Layout of calculation sections

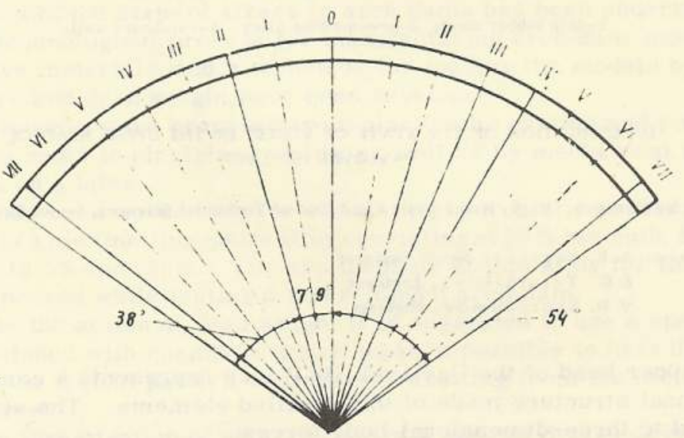


FIGURE 7. Stress distribution in arches:

I–development along axis: graphs for mean stress σ_θ; II–section A-A; graph for σ_θ; III–section B-B; graph for σ_θ; IV–section C-C; graph for σ_θ.

The lock walls basically behave as slabs embedded into the foundation plate and the concrete massif of the lift (breast) wall. The hydrostatic pressure on the side walls of a lock head is taken up to a considerable extent (up to 50%) by the lift wall, which acts at the same time as a rigid distance element between the side walls.

The existence of such a distance element creates a turning moment in the side walls, which increases in sections more distant from the foundation

slab. With such a turning moment in sections normal to the x axis, the external side of the wall will be subjected to tensile stresses while the inside part will be subjected to compressive stresses σ_y, which in the upper sections of the wall increase to about 4 kg/cm^2.

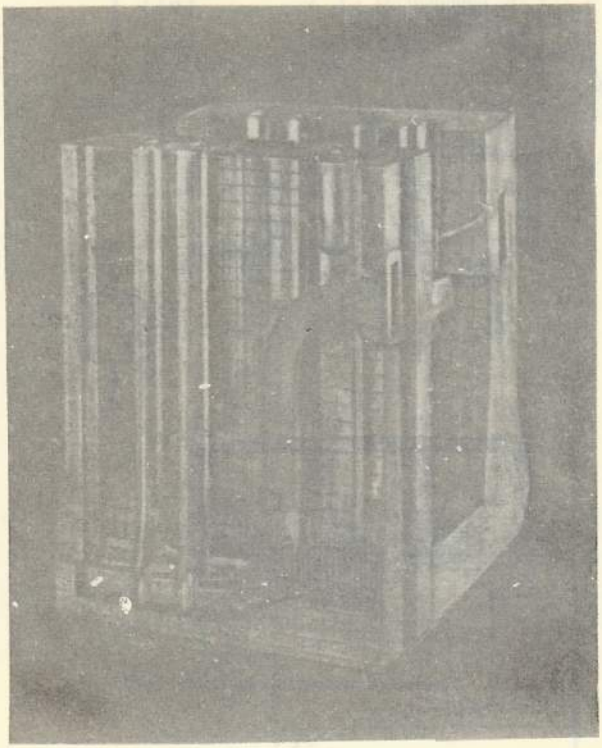


FIGURE 8. Lock model made of a glue-glycerine composition

All the sections considered are subjected to compressive stresses σ_y, which is due to the fact that the compressive stresses resulting from the dead weight of the lock-head wall markedly exceed the tensile stresses set up in them by the water pressure and backfill.

Characteristic stress diagrams for horizontal and vertical sections of one of the lock-head walls are given in Figures 9 and 10.

Thermal stressss in the structure were investigated by a method developed by Professor S. G. Gutman for solving one-dimensional problems of thermoelasticity.

The stresses were investigated only for the horizontal section of the lock head. In investigating the thermal stresses, the geometrical contour of the horizontal section was simplified.

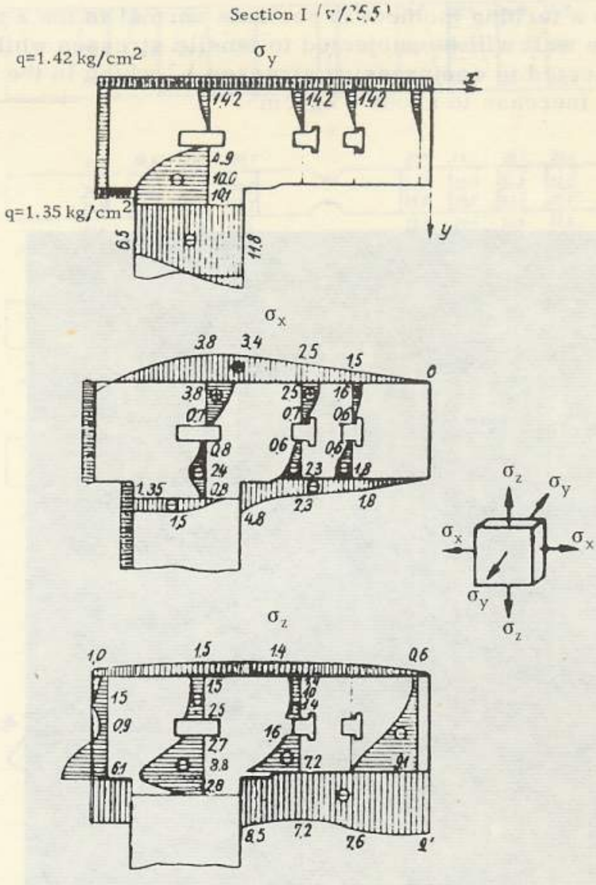


FIGURE 9. Diagram of stresses in a horizontal section of a lock-head wall

The temperature at the wetted perimeter of a horizontal section (the external contour up to the joint filler; internal apertures) was assumed to be 1°C. The temperature at the contour of the lock chamber was assumed to be zero. Hence, the temperature drop at the structure was assumed to be 1°C. In order to calculate the stresses for field conditions, the investigation results should be increased proportional to the actual value of the temperature drop.

In view of the symmetry of the horizontal section, only one half has been investigated.

The problems were solved in three successive stages:

At the first stage, the distribution of a temperature t and its gradients were determined on an electrical-analogy model using an electroconductive paper.

At the second stage, a particular solution to the problem, satisfying the conditions of internal thermoelasticity, was formed by means of distribution and gradients of temperature determined at the first stage of investigations.

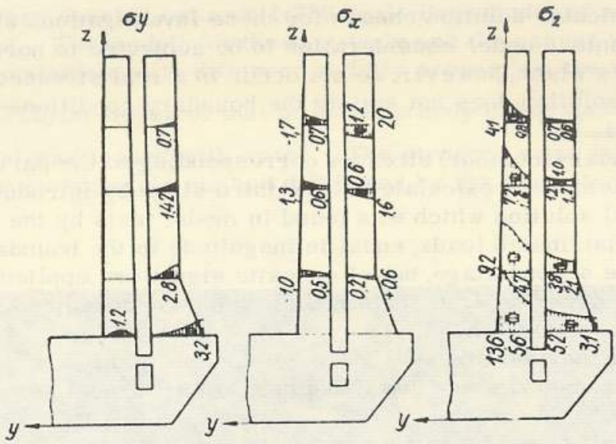


FIGURE 10. Diagram of stresses in the vertical section of one of the lock-head walls

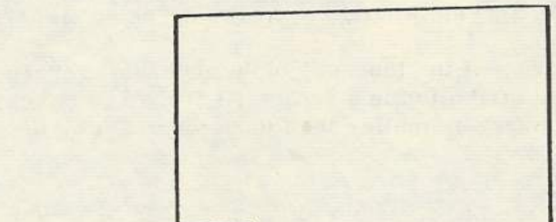


FIGURE 11. Diagrams of normal and tangential stresses

The particular solution chosen for these investigations shows the geometrical contour under consideration to be subjected to normal and tangential stresses which, however, do not occur in a real stressed structure. Hence, the solution does not satisfy the boundary conditions of the problem investigated.

The boundary (contour) stresses corresponding to the particular solution of the problem were calculated at the third stage by introducing an auxiliary general solution which was found in model tests by the tensometric method. Distributed loads, equal in magnitude to the boundary stresses obtained at the second stage, but of opposite sign, were applied to the contour of the glue-glycerine 1:50 scale model, which represented a horizontal section of the upper lock head.

The final thermal stresses were calculated as the sum of stresses obtained at the second stage and those determined experimentally at the third stage.

As a result of the investigation, the thermal stresses in the six sections close to the water passages and the lift wall, were determined.

The diagrams of normal stresses and of stresses tangential to the section are shown in Figure 11.

As can be seen, the maximum tensile stresses (during the winter) appear on a cooled surface at the junction of the lift wall with the lock-head lateral wall. These stresses have a value of + 1.6 kg/cm^2 per 1°C temperature drop, or 32 kg/cm^2 per 20°C temperature drop. This represents 80% of the value $E\alpha t = 40$ kg/cm^2 where $E = 2\times10^5$ kg/cm^2; $\alpha = 100\times10^{-7}$; $\mu = 0.18$; and $\Delta t = 20$°C)*.

Taking into account the tendency of the concrete to creep, and the variability of temperature distribution in a structure, it may be assumed that the actual stresses are somewhat smaller than those obtained by the investigation.

INVESTIGATION OF THE STATE OF STRESS IN THE ARCH DAM OF THE CHERKEISKOE HEP

Responsible for Research: N. S. Rozanov, Candidate of Technical Sciences, Senior Research Worker

Research by: Ya. G. Skomorovskii, Engineer

The Cherkeiskoe HEP dam is being erected on the Sulak River in a narrow canyon deeply cutting into limestone rocks.

The dam, designed by the Baku branch of the Gidroproekt, has a gravity-arch section, 185 m high, and a lower wedge-shaped concrete section, 55 m high. The dam is symmetrical in plan and bears against the upper, symmetric part of the canyon by means of abutments. Model tests of the state of stress in the arch dam of the Cherkeiskoe HEP, conducted at the OMIN laboratory, were instrumental in determining the stresses in the upstream and downstream dam faces, caused by water pressure, the weight of silt carried by the river and the dead weight of the structure.

* It should be noted that this magnitude of tensile stresses shows good agreement with the results of the approximate theoretical solution to the thermoelasticity problem investigated.

The tests were carried out on a 1:750 scale dam model of a glue-glycerine compound. The models for the dam body and the canyon walls (slopes) were made of materials with different rigidity properties, the ratio of the rigidity of the canyon slopes to that of the dam body being $\frac{E_{\text{can}}}{E_{\text{dam}}} = 0.5$ (Figure 12). The model was loaded with water. The stresses were investigated by a variant of the tensometric method developed by the laboratory.

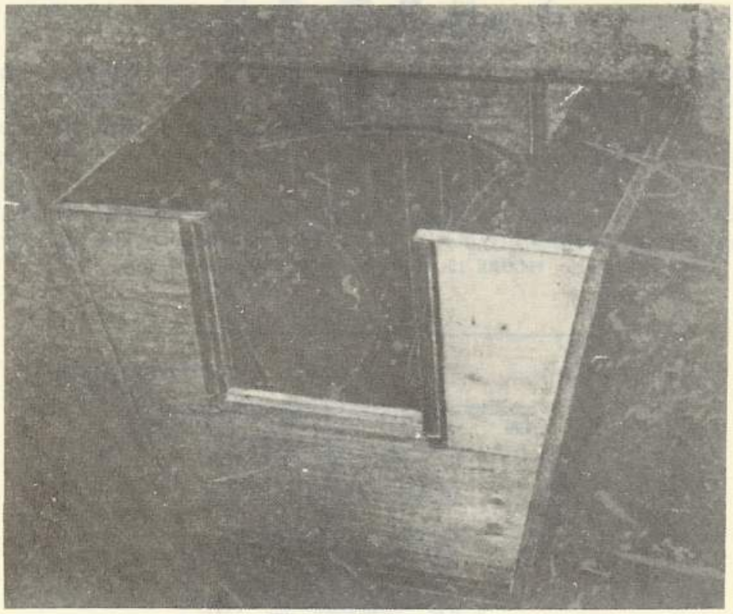


FIGURE 12. Model of an arch dam in a narrow canyon

These investigations made it possible to determine the normal stresses in a given section of the upstream and downstream faces of the dam. The upstream face was found to be subjected to compression resulting from the combined action of water pressure, dead weight and weight of silt. The maximum compressive stresses resulting from the hydrostatic pressure and the dead weight are: $\sigma_\theta = -51.4$ kg/cm^2 and $\sigma_\lambda = -63.7$ kg/cm^2. Stresses of such magnitude occur in the lower part of the central section of the face, at elevations close to the toe (wedge) (Figures 13 and 14). The maximum stresses caused by the weight of silt at the most stressed areas of the upstream face (for low elevations at the toe) are $\sigma_\theta = -7.0$ kg/cm^2 and $\sigma_\lambda = -6.4$ kg/cm^2.

On the downstream face the maximum compressive stresses caused by the hydrostatic pressure and the dead weight, occur in the vicinity of the abutments, approximately half way to the top and have the following values: $\sigma_\theta = -49.5$ kg/cm^2; $\sigma_\lambda = -27.5$ kg/cm^2. The maximum compressive

stresses resulting from the weight of silt are: $\sigma_\theta = -4.7$ kg/cm^2; $\sigma_\lambda = -27.5$ kg/cm^2.

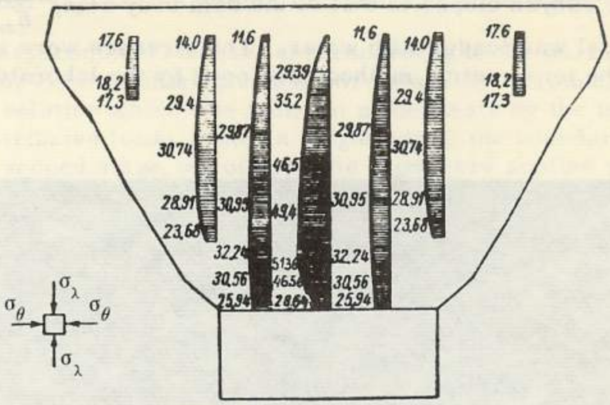


FIGURE 13. Upstream face. Diagrams of σ_θ

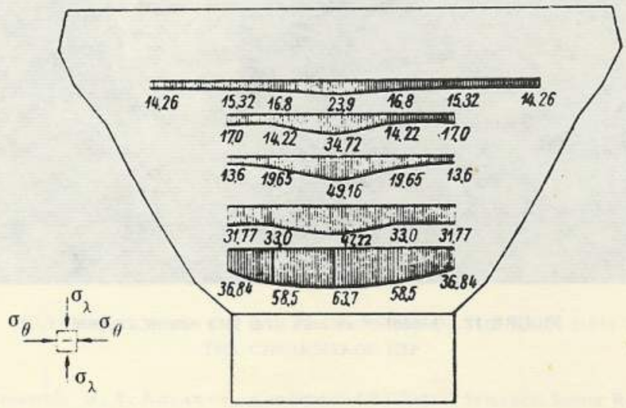


FIGURE 14. Downstream face. Diagrams of σ_λ

In the central part of the downstream face the stresses σ_θ and σ_λ caused by water pressure and the dead weight are 17.0 kg/cm^2 and 10.0 kg/cm^2, respectively. The stresses set up by the weight of silt are: $\sigma_\theta = 3$ kg/cm^2 and $\sigma_\lambda = 3$ kg/cm^2.

These investigations proved that the design of the Cherkeiskoe HEP arch dam conforms to the requirements set out for the construction of such structures, the maximum tensile and compression stresses not exceeding those accepted for reinforced concrete.

In conformity with the research schedule, the laboratory developed in 1958 a technique for applying photoelasticity methods in the study of arch dams with allowance for deformability of the dam base and the canyon slopes, and studied a number of structural elements of arch dams such as hinges, peripheral joints, etc.

INVESTIGATION OF THE STATE OF STRESS IN HYDRO STRUCTURES OF THE KRASNOYARSK HEP

Responsible for Research: N. S. Rozanov, Candidate of Technical Sciences, Senior Research Worker

Research Team: N. S. Dement'eva, Senior Engineer
I. A. Mikhailova, Engineer
L. P. Penkina, Acting Engineer

The laboratory of OMIN VNIIG carried out, during the years 1957 and 1958, investigations of the state of stress in various structures of the Krasnoyarsk-HEP dam. During 1958 the following design problems have been studied:

1) experimental investigation of stresses in a dam with expansion joints;

2) experimental investigation of stresses in an alternative design of the dam for a power plant constructed in two stages;

3) theoretical investigation of stresses in structures built-up on other substructures.

Investigation of the Krasnoyarsk-HEP dam erected with expansion joints was carried out on blocks 15 m wide and 114.5 m high having internal bays 3 m wide.

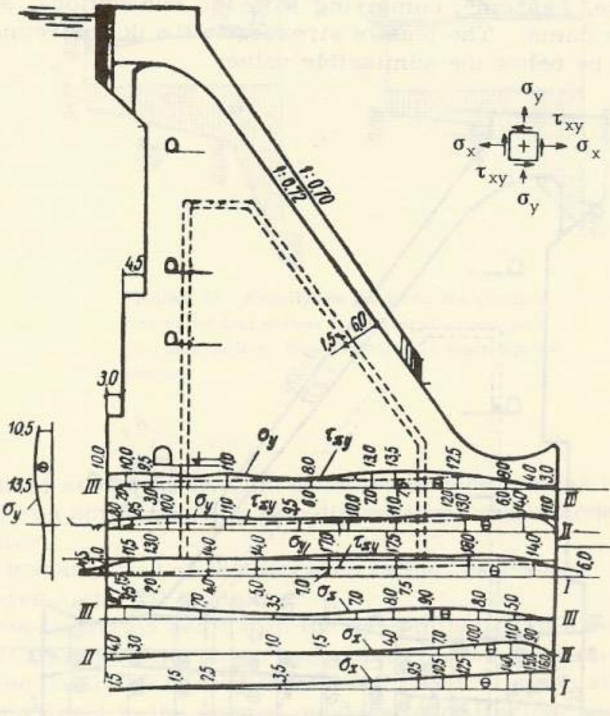


FIGURE 15. Diagrams of normal and tangential stresses

The experiments were conducted on reduced (small-scale) models, which permitted the determination of mean stresses through the block width.

For the purpose of these investigations it has been assumed that the rock foundation of the dam is fissured to a considerable extent; consequently, the stress-strain modulus of the rock was assumed to be smaller by a factor of 1.5 to 2 than that of the concrete masonry. Therefore, the models were made of two grades of glue-glycerine composition with a given ratio of stress-strain moduli.

The stresses in the block were determined for the following load scheme:

a) loads from the dead weight of the structure;

b) combined action of dead weight and water pressure on the upstream face;

c) combined action of the same forces and of the horizontal seismic forces.

The 1:250 scale dam model was made of glue-glycerine composition, and the stresses in the model were determined by the tensometric method.

The investigation made it possible to determine the stresses in typical horizontal and vertical sections of the dam block.

The results obtained show that the state of stress of the Krasnoyarsk-HEP dam built with expansion joints is satisfactory under all the investigated load combinations. Stresses σ_y taken as an average over the dam body in the dangerous sections of the upstream face are compressive, their minimum value, 3 kg/cm^2, complying with the conventional strength requirements for dams. The tensile stresses in the downstream face of the dam proved to be below the admissible value.

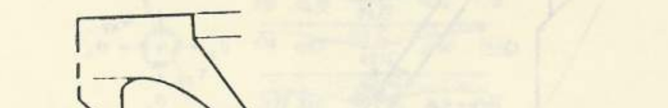


FIGURE 16. Action of the dead-weight load

In the investigation of the state of stress of the dam for the HEP erected in two stages, it has been assumed that initially the dam will be erected up to a certain intermediary height and then the storage reservoir will be filled up to this elevation. This construction method permits the power plant to be put into operation while construction is still going on. Erection of the dam to its full height will be completed subsequently, the concrete for the part yet to be finished being placed on the downstream side of the dam being under load. The basic dimensions of the dam for the "two-stage" HEP are the same as for the monolithic (one-stage) variant of the Krasnoyarsk HEP dam.

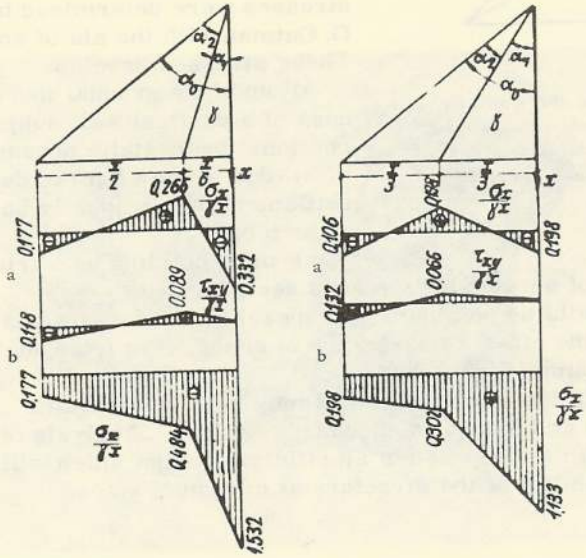


FIGURE 17. Calculation graphs for the computation of residual stresses in a triangular cross-section dam built-up from subsequently superimposed sections

The state of stress in the dam under consideration has been tested experimentally on corresponding two-dimensional scale models by the tensometric method.

For the investigation of the stresses in the "two-stage" dam the laboratory developed a special technique.

These investigations dealt with the determination of stresses in a dam erected in two stages, subjected to hydrostatic pressure, with the storage reservoir filled partly up to the elevation of the first stage and increased upon raising the level to the normal pondage level [further abbreviated in this text by NPL] acting on the full profile of the dam. The stresses thus formed were added to the stresses produced by the dead weight. The tests were carried out by the tensometric method on 1:250 scale models made of glue-glycerine composition.

The glue-glycerine composition for the foundation model had a modulus $E = 0.6$ kg/cm^2, while that used for the model of the structure had a modulus $E = 1.2$ kg/cm^2. For comparison Figures 15 and 16 show stress diagrams for the one-stage and two-stage dams, respectively.

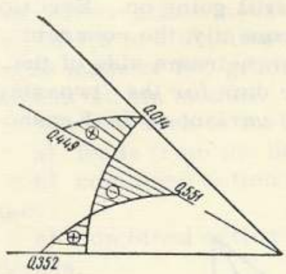


FIGURE 18. Calculation graphs for the determination of residual stresses resulting from the shrinkage of concrete in triangular and trapezoidal dams

The problem of the effect of the subsequent-stages method of erection and loading on the state of stress in massive structures has been theoretically investigated by methods of the theory of elasticity. Calculation graphs used for computation of residual stresses were determined by Professor S. G. Gutman with the aid of analytical solutions. These stresses develop:

a) upon the gradual increase in the thickness of a vertical wall subjected to unidirectional hydrostatic pressure;

b) due to the effect of dead weight and settlement of a gradually built-up rectangular block;

c) upon building up a triangular cross-section dam of successively placed sections;

d) upon building up a dam of trapezoidal cross sections;

e) under the effect of shrinkage of concrete in triangular-profile and trapezoidal dams (Figure 18).

The calculation graphs obtained may be used for static computation of dams erected and loaded in successive stages. Analysis of these relationships permits the selection of an efficient design which will ensure the maximum stability of the structure at minimum sizes.

INVESTIGATION OF THE STATE OF STRESS IN THE SUBMERGED [CONCRETE] SECTIONS OF POWERHOUSES OF RUN-OF-RIVER HEPS

Responsible for Research: O. V. Novikova, Candidate of Physical and Mathematical Sciences, Senior Research Worker

The computation of the state of stress in the submerged structures of a powerhouse is greatly complicated owing to the intricate shape of these structures. Special experimental research was therefore needed to determine the most stressed areas and to ensure the most effective reinforcement design.

Since the individual elements of the structure behave as a whole (monolithic) structure, the experiments had to be conducted on three-dimensional models of the submerged massif. Considerable difficulties arose both in the preparation of intricately shaped models and in the simulation of the complex pattern of the acting forces.

The model to be tested simulated that portion of the powerhouse accommodating two hydro-turbine units. The foundation was modeled as a deformable body, the ratio of moduli of elasticity for materials of the structure and the foundation being 154.

The tests were conducted by the photoelastic method using the frozen-stress technique. The 1:800 scale models were made of epoxy resins.

In 1958 a method was developed for manufacturing three-dimensional, intricately shaped models. The models were assembled from separate compartments joined by a special adhesive. The various sections of the model were cut out from a block of the starting material.

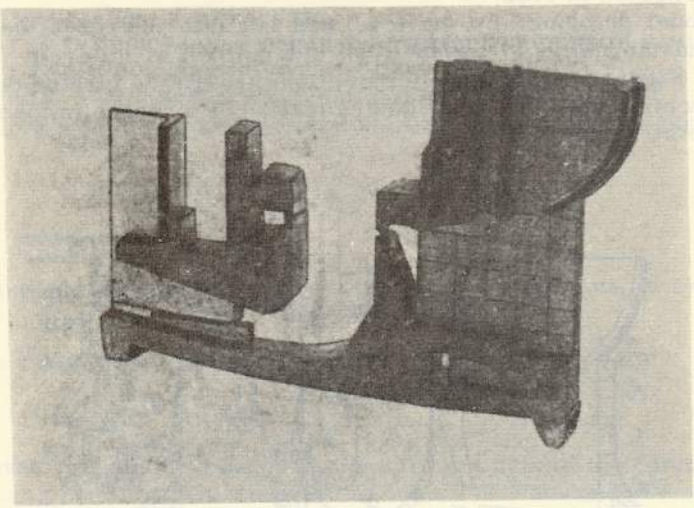


FIGURE 19. Model of microsection through the turbine center line

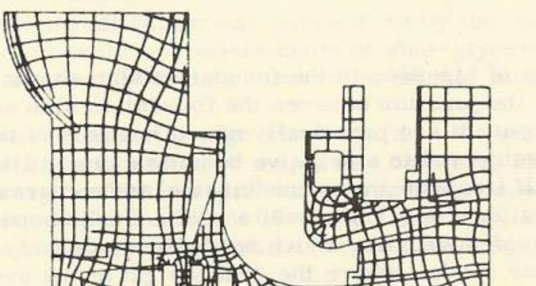


FIGURE 20. Lines of normal stresses acting in the plane of the microsection

Special experiments dealt with the selection of such a material for the foundation, that would retain the necessary mechanical properties at a testing temperature of 120°C. A special spongy (microporous) rubber with an elasticity modulus $E_{\text{T}} = 0.9$ kg/cm^2 at 120°C was found to be the most suitable material.

The action of dead weight of the intricately shaped structure was simulated on a large-size centrifuge especially designed for this purpose. Stresses equivalent to those resulting from the dead weight of the structure were simulated by the centrifugal force.

A special technique has been developed to simulate the hydrostatic pressure by means of a eutectic alloy with density $\gamma = 10$ g/cm^3. The alloy melts at the test temperature, thus creating hydrostatic pressure on the model. Stresses produced by the dead weight of a submerged structure were investigated in 1958.

The model deformed by the load was cut into microsections and the normal and tangential stresses were determined at different points and sections. Figure 19 shows a microsection through the turbine center-line. Special attention was paid to the determination of stresses in the foundation slab of the structure.

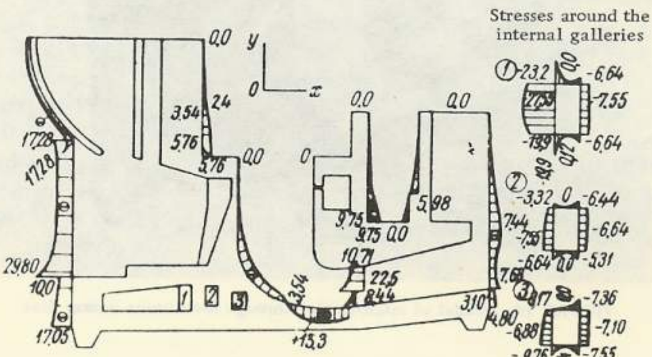


FIGURE 21. Normal stresses σ kg/cm^2 along the free contour

The analysis of stresses in the foundation slab shows that the tangential stresses along the junction between the foundation slab and the deformable rock layer beneath it are practically nil. That part of the foundation below the turbine is subjected to a negative bending moment like a beam stretched in its upper half (from the water-conduit side) and compressed in its lower half (from the foundation side) (Figures 20 and 21). Such a bending is evidently characteristic of a soft foundation which settles considerably in the areas adjacent to the water conduit where the dead weight of the overlying part is greater than that acting upon the areas below the water conduit.

Microsections through the stream path show the following stress pattern: stresses σ_y along the vertical reach the maximum value at the base in the area of the upstream face, being 33.6 kg/cm^2. Tensile stresses around the contour of the water conduit (draft tube) below the turbine reach 13.3 kg/cm^2.

GUIDE FOR PHOTOELASTIC INVESTIGATIONS ON HYDRO-STRUCTURE MODELS

Responsible for Research: O. V. Novikova, Candidate of Phys. and Math. Sciences, Senior Research Worker

The aim of the work has been to systematize the experience gained by the laboratory in the field of photoelastic investigations on stresses in hydro structures.

The guide deals with the following problems of model tests: effect of dead weight; water and silt pressure; ice pressure; uplift pressure resulting from ground-water seepage as well as the effect of seismic forces. Certain specific problems concerning the investigation of two-and three-dimensional hydro structures have also been included.

The guide is divided into five chapters:

Chapter I – General principles, and techniques for simulating different types of loads acting on a structure.

Chapter II – Methods of investigation, and description of equipment.

Chapter III – Photoelastic investigation.

Chapter IV – Processing of investigation results.

Chapter V - Method for computation of stresses from the obtained optical pattern.

INVESTIGATION OF THE STATE OF STRESS OF THE SANMENHSIA HEP DAM* AND ITS STRUCTURAL ELEMENTS

Responsible for Research: R. G. Konstantinova, Senior Engineer

Research by: I. A. Mikhailova, Engineer

The investigation of stresses in the spillway dam of the Sanmenhsia HEP on the Hwang Ho River was intended to supply necessary information for the working design. The design height of the dam is 88.5 m. The design provides for stationary and temporary water-discharge openings in the dam body. A deep tectonic fracture filled with detritous rock runs through the dam site. Owing to the large amount of sediment load carried by the Hwang Ho River the unit weight of water varies from 1 t/m^3 at the water surface to 1.5 t/m^3 at the dam base.

The experimental investigation was carried out by the tensometric method on reduced two-dimensional models made of glue-glycerine composition, and by the photoelastic methods on a three-dimensional model made of ED-6 epoxy resin.

To select the most efficient design with regard to the static behavior, an investigation of the state of stress in a spillway dam with four alternative designs of temporary and permanent water-discharge openings was carried out. The tests were conducted on 1 : 200 scale models made of glue-glycerine composition with the ratio between the moduli of elasticity $\frac{E_c}{E_w} = 1.7 - 2$.

* [On the Hwang Ho River in China.]

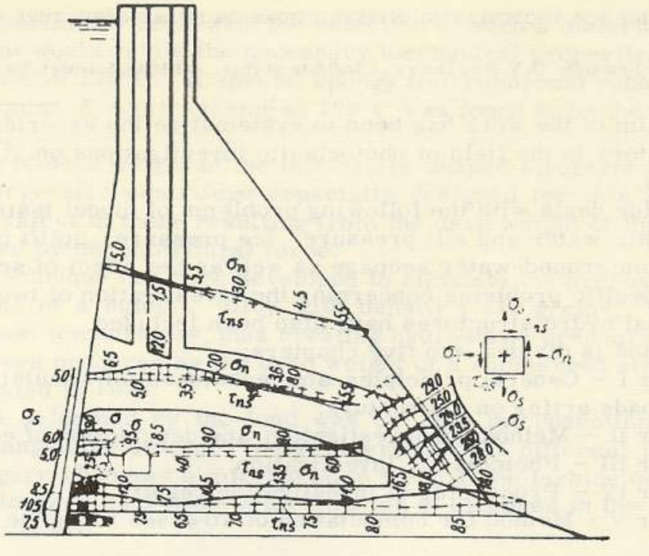


FIGURE 22. Diagrams of normal σ_s and tangential τ_{ns} stresses (design II)

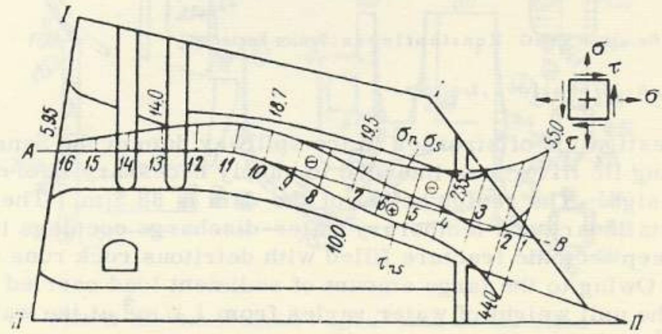


FIGURE 23. Diagrams σ_n, σ_s, τ_{ns}. The central pier of a block

Further investigations of the selected dam design were intended only for determining the stresses in the area of water-discharge openings, and were carried out on a three-dimensional model made of Ed-6 epoxy resin. The investigations were instrumental for the detailed study of the stress pattern in water-discharge outlets and in the central part of the dam.

After processing the data obtained, diagrams of the normal and tangential stresses, σ_n, σ_s and τ_{ns} respectively, have been plotted for preliminarily chosen sections of the structure. In selecting the most efficient design of water-

discharge outlets, the distribution of normal stresses σ_n on the upstream face in the area of the water-discharge outlets was taken into account. The analysis of test results shows that the most efficient design with regard to the static behavior of the structure is that providing for discharge outlets slightly widening toward the downstream face of a dam.

The results of investigation of stresses in the zone of water-discharge outlets are shown in Figures 22 and 23, in the form of diagrams for normal and tangential stresses σ_n, σ_s and τ_{ns} respectively. Analysis of these diagrams shows that principal tensile stresses occur in the dam piers at the downstream face. It is therefore recommended to reinforce these areas of the piers.

MODEL TEST OF THE STATE OF STRESS OF THE BRATSK HEP DAM

Responsible for Research: V. N. Mokrushin, Senior Engineer

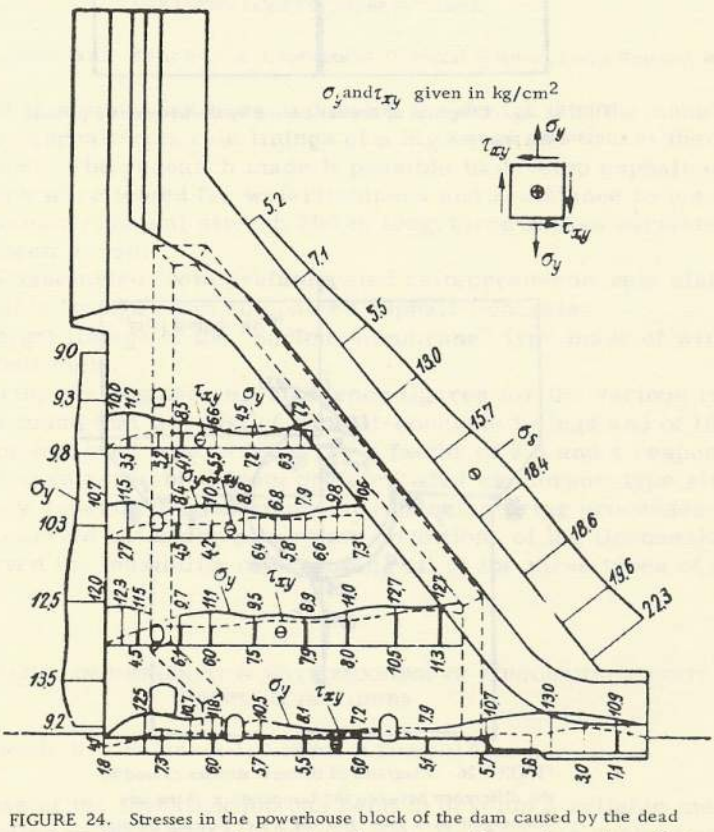


FIGURE 24. Stresses in the powerhouse block of the dam caused by the dead weight of the structure and the hydrostatic pressure acting on the upstream face

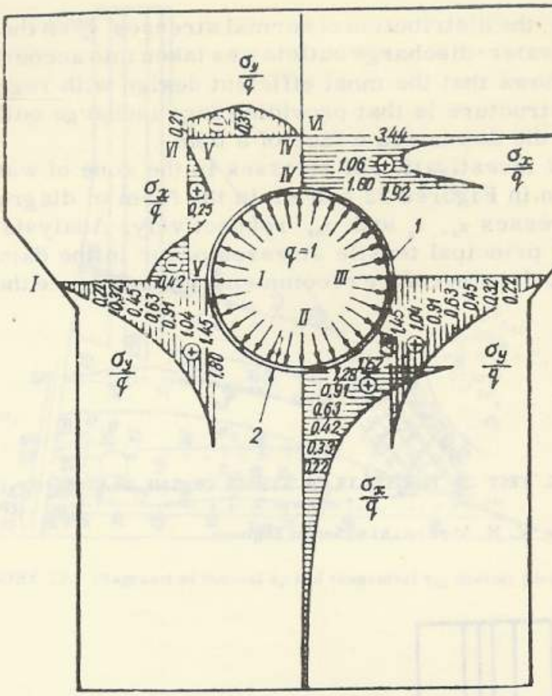


FIGURE 25. Diagrams of stresses caused by the hydrostatic pressure inside the penstock

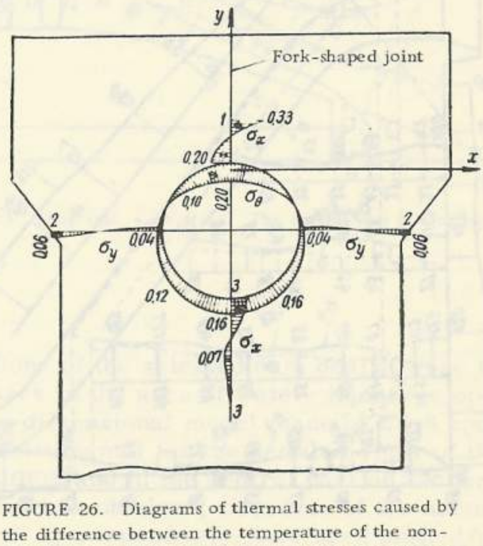


FIGURE 26. Diagrams of thermal stresses caused by the difference between the temperature of the non-submerged dam face and the water flowing through the penstock

The aim of these investigations involving the use of models has been to determine the state of stress of the powerhouse block (bay) of the Bratsk HEP dam.

The work was divided into three stages. The first stage dealt with the determination of the stresses caused by the dead weight and the water pressure on the upstream face of the dam (Figure 24).

The second stage covered the study of stresses in a section of the powerhouse block perpendicular to the penstock axis and resulting from the hydrostatic pressure within the penstock (Figure 25).

Diagrams of thermal stresses caused by the difference between the temperatures of the nonsubmerged face of the dam and the water flowing through the penstock (Figure 26) have been obtained at the third stage.

VNIIG WATERPROOFING LABORATORY IMENI B. E. VEDENEEV

Head: Professor P. D. Glebov, Doctor of Technical Sciences, Honored Scientist of the R. S. F. S. R.

INVESTIGATION OF ASPHALT-CONCRETE LININGS FOR THE NORTHERN DONETS-DONBASS CANAL

Responsible for Research: S. N. Popchenko, Candidate of Technical Sciences, Senior Research Worker

The aim of this study has been to develop the design and the manufacturing technology for asphalt-concrete linings of a big canal, and to test them under field conditions. The research made it possible to develop asphalt-concrete mixtures which were tested for watertightness and resistance to ice pressure. For an experimental stretch 600 m long, three design variants for linings have been tested:

1) linings assembled from prefabricated reinforced-concrete slabs;
2) monolithic linings from compacted asphalt-concrete;
3) lightweight linings of the "buried-membrane" type made of wire-reinforced asphalt mats.

By comparing the engineering-efficiency figures for the various types of linings it was found that the cost of asphalt-concrete linings and of linings of the "buried-membrane" type is lower, by a factor of 2.5 and 4 respectively, than that of linings assembled from prefabricated membrane-type slabs.

Preliminary experiments for improving manufacturing processes for linings were carried out under pilot-plant conditions of the Donbasskanalstroi and proved the feasibility of producing all of the three types of lining.

INVESTIGATION OF IMPREGNATION WATERPROOFINGS OF REINFORCED-CONCRETE TUNNEL-SECTION LINERS

Responsible for Research: N. S. Pokrovskii, Candidate of Technical Sciences

The purpose of the investigation has been to develop a reliable method for waterproofing precast reinforced-concrete liners for a subway underground tunnel. It has been intended to study the impregnation of concrete

with bitumen and certain other organic compounds as well as to investigate cold-setting epoxy resins used for protective-paint coatings.

When starting the investigation in 1957 it was found that, owing to the considerable density of the concrete used by the Mosmetrostroi, an open-bath impregnation with a BN-III bitumen requires about 50 hours to attain an impregnation depth of 20 mm. The method had to be rejected because of low engineering efficiency and attention was therefore paid mainly to the investigation of new impregnating agents, such as petrolatum and ozocerite. The use of these materials permits a considerable reduction of impregnation time and production cost.

Protective epoxy-resin coatings applied at the factory on the liners, proved unsuccessful, owing to damage during transportation and subsequent assembly of the tunnel section.

In 1958 problems concerning protective coatings were not studied further. The investigations conducted dealt with the properties of petrolatum-impregnated concrete, the impregnation technology and testing under working conditions. As a result of the tests it was found that petrolatum impregnations almost do not affect the strength properties of a concrete neither under static nor dynamic loads. They significantly improved frost and corrosion resistance as well as watertightness, but, on the other hand, reduced the adhesion of concrete to smooth (round) reinforcement bars.

To improve watertightness and corrosion resistance, an addition to petrolatum is recommended of 15 % (by weight of the mixture) of paraffin.

Investigations of the impregnation techniques dealt with the development of optimum working conditions for the various operation stages, especially of drying and impregnation. A forced air circulation inside the drying chambers is recommended. Though no reduction of drying time was achieved, the air circulation ensured a uniform drying and the elimination of large thermal stresses.

Impregnation should preferably be carried out in autoclaves since here the process takes only a tenth of the time of dipping in open baths. The first factory experiments were carried out in open baths on 188×100×25 cm slabs each weighing 1200 kg and made of "grade 400" concrete. The results obtained by laboratory investigation of the drying process and of the behavior of petrolatum-impregnated concrete were confirmed experimentally. However, it was found that the watertightness of impregnated slabs should be improved; watertightness is impaired by the existence of large holes and cracks within the concrete mass, which cannot be properly insulated by impregnation. The defects in concrete are due to the high stiffness (W/C ratio = 0.3) of the mix and to shortcomings in the manufacture of the tunnel sections.

To ensure higher watertightness of the concrete liners it is necessary to continue the investigation throughout 1959, particularly with respect to the development of a proper impregnation technique and application of ozocerite for the impregnation.

The investigation report and instructions have been passed on to the institutions concerned.

DEVELOPMENT OF METHODS FOR RECONDITIONING OF OLD, AND THE DESIGN OF NEW, ASPHALT FILLERS IN EXPANSION JOINTS OF THE STRUCTURES OF THE TULOMA HEP

Responsible for Research: A. N. Khudyakov, Junior Research Worker

In view of seepage phenomena noticed in the expansion joints of the headwater face of the Tuloma HEP powerhouse and in the joints of the cable galleries, investigations have been carried out to develop methods for restoring old and designing new asphalt fillers for this type of joints.

The investigation schedule included:

1) inspection of the powerhouse and cable gallery; selection from the file of available technical documentation necessary for the design of fillers;

2) development of methods for the redesign of fillers and sealings;

3) recommendations for proper work planning, technology and indications concerning the necessary equipment and materials;

4) preparation of the technical report on the accomplished job;

5) draft of instructions for the execution of waterproofing work.

In the course of 1957 the first, and parts of the second and the third points of the above schedule have been accomplished.

As a result of investigations a method for reconditioning the seals has been developed, consisting in the installation of new fillers in the joints of the powerhouse and in the revision of joints in the cable gallery. The new fillers are made by drilling boreholes 18-20 cm in diameter and filling them with a hot asphalt compound under pressure through a pipe running down through the center of the borehole.

The control of the fillers in the joints in the cable gallery consists in opening and, if necessary, repairing the seals and fillers. In order to check the possibility of feeding the hot asphalt compound into the flooded holes, laboratory tests were carried out using a seal model filled with water.

Finally, a preliminary specification of equipment and materials necessary for execution of the reconditioning work has been compiled.

The work on the second and third points of the investigation schedule continued during 1958. Equipment, drawings, working schedule, and calculations connected with pumping the asphalt compound into the boreholes have been prepared. The technical report mentioned under (4) has been prepared in 1958 and given to the customer.

The management of the HEP not yet being prepared for the execution of the work, the relevant instructions will be given in 1959.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR PLANNING AND EXECUTION OF CONSTRUCTION WORK

Head: I. E. Kartelev, Candidate of Technical Sciences, Senior Scientific Worker

ON THE FEASIBILITY AND ENGINEERING EFFICIENCY OF ERECTING THE FORE APRON BY DEPOSITING CLAYEY SOIL IN THE WATER, AT THE CONSTRUCTION OF THE STALINGRAD HYDRO DEVELOPMENT

Responsible for Research: P. I. Volodenkov, Group Engineer

The investigation has been commissioned by the construction agency. The investigation schedule provided for the systematization of the

experience in depositing clayey soils in water, participation in the experimental work, and drafting instructions for the erection of the fore apron. Experimental work has been conducted at the construction site in conformity with temporary technical specifications.

The volume of clay depositing provided for one working plot was 7500 to 10,000 m^3, with a depositing rate of 1000 to 2000 m^3 per shift.

At one experimental plot the fore apron was erected in one layer with a design height of 2 to 4 m. At the second plot the apron was erected in two layers up to 1 m high each.

Quality of work was checked by the laboratories of the Stalingradgidrostroi, Gidroproekt and VNIIG, core samples being taken periodically from boreholes after completion of clay depositing.

The laboratory tests proved that, as a result of physical, mechanical and chemical processes, the soil deposited in water undergoes substantial changes. In the course of time the moisture content and density of the deposited soil layer are leveled out and its compactness increases. The leveling of moisture and increase in compactness proceed from bottom to top across the whole thickness of the fill.

The effect of time on the moisture content and compactness of the fill is given in Tables 1 and 2.

The investigation results proved that after a short period (two months) clayey soil deposits in water acquire a compactness sufficient for watertight elements of hydro structures and, still more, for earth dams.

The seepage coefficients for soils having a density of 1.55 t/m^3 are: $k = 10^{-7}$ to 10^{-8} cm/sec, i.e. considerably less than those taken into account in the design ($k = 5 \times 10^{-5}$ cm/sec).

The systematization of experience in depositing clayey soil in water and the experimental work on the construction of the fore apron parallel to the whole length of the spillway dam of the Stalingrad hydro development, permitted the following conclusions to be drawn:

1. Depositing of clayey soil in water may be considered as a further perfection of earth-fill methods, numerous shortcomings typical of the rolled-layer and hydraulic-fill methods being eliminated.

2. The method of soil depositing in water ensures formation of a soil layer of optimum moisture content which in turn ensures a maximum compactness of the soil in the structure.

3. This method permits permanent control of the state of the soil layer, thus aiding it in becoming subsequently a monolithic body.

4. Clayey soil deposited in water acquires a reliable bond both with the whole structure and with its component elements.

5. In erecting structures by the above method, the requirements imposed on the quarries become less stringent, as it is possible to use filling soil of less uniform moisture content and granular composition.

6. The above method favors the maximum mechanization of construction operations, thus ensuring a high work rate over a great length of the layer to be filled; moreover, the volume of work for quality control of the structures is considerably simplified and reduced.

7. The existing technical specifications for the construction of earth structures by the above method are now obsolete, and it is necessary to prepare new, up-to-date specifications which can be prepared on the basis of ample information available.

The construction of the fore apron for the Stalingrad hydro development by the above method lasted three months and resulted in savings of 1 million rubles.

TABLE 1

Data on the first experimental plot

No	Duration of laboratory test	Moisture content, %			Bulk density of the soil matrix, t/m^3		
		min.	max.	average	min.	max.	average
1	After 5 days (Stalingrad-gidrostroi)	15.6	35.0	22.4	1.27	1.64	1.47
2	After 15 days (Gidro-proekt)	15.8	34.1	23.7	1.38	1.76	1.53
3	After 30 days (Stalingrad-gidrostroi)	12.5	32.5	22.9	1.41	1.83	1.60
4	After 45 days (Gidro-proekt)	14.5	34.8	20.8	1.40	1.95	1.66

TABLE 2

Data on the second experimental plot

No	Duration of laboratory test	Moisture content, %			Bulk density of the soil matrix, t/m^3		
		min.	max.	average	min.	max.	average
1	After 10 days (Stalingrad-gidrostroi)	17.5	37.2	24.6	1.29	1.72	1.45
2	After 30 days (VNIIG imeni B. E. Vedeneev)	18.66	26.18	22.16	1.54	1.58	1.56
3	After 30 days (Gidro-proekt)	14.4	29.2	21.2	1.47	1.76	1.64
4	After 60 days (Stalingrad-gidrostroi)	8.7	24.3	16.7	1.53	1.85	1.69

SYSTEMATIZATION OF EXPERIENCE IN APPLYING PLAIN AND REINFORCED-CONCRETE FACING SLABS FOR HIGH, MASSIVE HYDRO STRUCTURES

Responsible for Research: S. I. Zubovich, Candidate of Technical Sciences, Senior Research Worker

The purpose of the work has been to investigate the suitability of existing designs of reinforced-concrete facing slabs for facing and formwork of the Sanmenhsia HEP structures.

Analysis of China-made facing slabs showed that their application for nonreinforced concrete structures similar to those of the Sanmenhsia HEP is not advisable.

Facing blocks used abroad for the same purpose are of small sizes, which fact involves greater length of the connecting joints, and therefore makes them unsuitable.

Studies were therefore initiated on the design of new types of facing slabs being at the same time a protective structure for the basic concrete massif and the formwork during the whole period of concreting of the structural blocks. Such protective elements are massive, ribbed concrete slabs.

The problem of properly sealing the joints between the slabs is far from being completely solved. Therefore, studies were carried out permitting a new technique to be developed consisting in pumping a cement grout into special recesses cut out along the edges of slabs. The experiments proved that this technique is simple in operation and ensures watertightness under a pressure exceeding 10 atm.

The use of vibration-ground cement for the manufacture of facing slabs was studied with a view to reducing their production cost. It was found that, as a result of additional grinding of the cement in a vibration mill (for 10 to 20 minutes), the 24-hour strength of concrete is increased to ~ 200 kg/cm^2. This permits the steam-curing operations to be eliminated, resulting in a marked reduction in production cost of facing slabs.

In addition, more research was carried out to improve the technique of sealing the joints between thin-wall slabs. It was found that a preliminary treatment of joints with a mixture of glue No. 88 and bitumen, followed by calking of the joints with cement grout, considerably increases the watertightness of the joints.

The final reports include recommendations on selection of suitable types of facing slabs for different applications as well as a description of their production and assembly methods.

NONREINFORCED CONCRETE FACING SLABS FOR MASSIVE STRUCTURES

Responsible for Research: S. I. Zubovich, Candidate of Technical Sciences, Senior Research Worker

The purpose of the investigation has been to determine the most suitable type of concrete formwork that could simultaneously be used as a facing for the concrete structures to be erected.

The study consisted in generalizing Soviet experience in application of reinforced-concrete facing slabs; foreign experience in using shell slabs for facing the dams; research material on the development of new slab types and production methods. Additional tests were carried out to find proper methods of sealing the joints and ensuring the monolithic character of the joined elements.

From data of available technical literature and field tests it was established that the further application of reinforced-concrete slabs of the existing design is not suitable since they do not ensure protection of the basic concrete structure; moreover, their production cost is 2 to 3 times higher than that of the basic monolithic concrete structure. Apart from this, the facing slabs have to be reinforced, which is not justified as, in this case, the reinforcement does not function as a load-carrying element. The use of facing slabs for massive concrete structures requires special attachment fixtures such as reinforcing frames, ties, etc.

According to foreign experience in the use of concrete facing-slab formwork, nonreinforced concrete slabs may be successfully used for facings, thus leading to a considerable simplification of their production process.

From the analysis of the effectiveness in using reinforced-concrete facing slabs and reinforcing panel elements recently introduced into practice, the conclusion may be drawn that reinforcing panels are the most suitable material for the erection of reinforced-concrete structures, while nonreinforced facing slabs should be preferred for massive concrete structures.

In the search for new designs of facing slabs for nonreinforced structures, it was found that mass concrete slabs are more effective than reinforced-concrete facing slabs. It should also be mentioned that the process of production and assembly of such facing slabs is more simple. Thus, it was found that 60 to 70% of the design strength of concrete may be obtained as early as 48 hours after placing, provided that 20% of the cement used be subjected to additional vibration grinding for five minutes and 3% of gypsum be added.

Moreover, if the hardening of concrete is ensured at a temperature of 30 - 40°C, finished concrete products of required strength can be obtained without steam curing, already 24 hours after placing.

The sealing bond between the slabs and the base body to which they are attached was tested on model blocks 1.5×2.5m in size, faced with slabs of dimensions reduced to the model scale, by pumping water at a pressure of 10 atm into the recesses arranged along the edge of the slabs. The joints were found to be sufficiently watertight, no leakages appearing at the test pressure.

Additional tests were carried out of the tightness of the joints between the slabs proper. Different bituminous compounds, a mixture of acidol and naphthenic soaps, various mixtures of glue No. 88 with and without cement, were tested for this purpose. However, it should be mentioned that, due to the insufficient deformability of the tested materials and compounds, the experiments failed to yield favorable results. Thus, in testing the joints sealed with the above compounds, it was found that a joint at the age of one month is watertight at a water pressure of 4, 6 and 8 atm. However, after 3 to 6 months the joint started to leak at a pressure as low as one atm. Evidently the failure is due to shrinkage of the sealing material. Therefore, a further investigation of materials ensuring the required watertightness is scheduled for the near future.

Economic calculations showed that the production cost of mass concrete slabs is only slightly higher than that of cast monolithic concrete (cost of slabs is 350 to 360 rubles/m, cost of monolithic concrete - 281 rubles/m).

The problem of selecting the optimum design of facing slabs of the two- or three-dimensional type depends on the specific conditions of the structures and, especially, upon climatic conditions.

It was proved by special calculation of thermal conditions that in a severe climate, like that of the Bratsk HEP region, it is preferable to use three-dimensional box-type slabs, whereas for a milder climate flat concrete slabs may successfully be used.

However, neither a final conclusion about the proposed design for mass concrete slabs, nor a selection of the most suitable type are possible unless their behavior under working conditions is checked.

TNISGEI IMENI A. V. VINTER LABORATORY FOR ENGINEERING GEOLOGY

Head: L. N. Lomize, Candidate of Technical Sciences, Senior Research Worker

INDICATIONS FOR THE DESIGN OF CONTROL AND MEASURING EQUIPMENT FOR THE NAMAKHVANI HEP ARCH DAM

Responsible for Research: A. N. Chizh, Junior Research Worker

The purpose of the work has been:

1) to prepare suggestions for the design of control and measuring equipment for the Namakhvani HEP arch dam;

2) to recommend types of control and measuring instruments taking into account Soviet and foreign experience, and also to draw an approximate scheme for their layout.

The work consisted in the following stages:

1) description and critical analysis of methods used for measuring stresses and deformations in concrete in order to serve as a guide for the selection of the most effective method for testing stresses in concrete;

2) suggestions for selection of strain gages, piezodynamometers, etc. and on the applicability of special strain gages for measuring stresses in rocks;

3) description of the progress of tests on arch dams and of control and measuring instruments for:

a) measuring stresses and deformations in concrete and rock; b) measuring pore pressure in concrete and rock, determination of seepage flow in masonry and at the dam base; c) determination of deformations in expansion joints; d) determination of the overturning moment of the dam; e) measuring deformations of the reservoir banks of the settlement of the dam base, investigating bedload movement in rivers and morphological changes in the reservoir banks;

4) technical specification for embedding and mounting the control and measuring instruments;

As a result of these investigations, the following conclusions were drawn:

1) the tensometric method of stress determination yields the best picture on the state of stress in concrete, if the following instruments are used:

a) a set of primary strain gages of the star or rosette type; b) cones for measuring temperature contraction and expansion; c) special pile-type gages for determination of E and μ of concrete and of its creep behavior; d) instruments for determining pore pressure [in concrete or rocks], thermometers and hygrometers;

2) for a better processing of results, standard types of instruments should be used; the strain sensitivity of strain gages should correspond to the design strength of concrete;

3) in order to facilitate the circulation of moisture in the pile-type gages used for determining E, μ and creep behavior of concrete, the supporting frame of the gages should be perforated;

4) the small-range Carlson-type and other types of strain gages are not sensitive enough for detection of creep, settlement and swelling of concrete. This is due to the interference of the coarse aggregates in the concrete mix. Therefore, these gages mainly react to those strains

which occur in the coarse aggregate itself, i. e. they become less sensitive to the creep and other deformations;

5) to avoid distortion of readings of the pressure-gage-type piezometers, the pressure gages should be installed with their connecting nipples upward, while the whole measuring system, including the Bourdon spring, should be filled with water.

Control and measurements at the Namakhvani HEP arch dam will permit a comparison between the operational behavior of the dam and the design assumptions as well as the model-testing results.

This study has been used for the design of Namakhvani HEP. The main conclusions of this work will also be made available to the designers of the Ingur arch dam.

TNISGEI IMENI A. V. VINTER HYDRAULIC ENGINEERING LABORATORY

Head: L. G. Gvelesiani, Candidate of Technical Sciences, Lecturer

FIELD AND LABORATORY INVESTIGATIONS OF OPERATIONAL EFFICIENCY AND DESIGN OF THE SCREEN FOR LEVELING OUT FLOW-VELOCITY FLUCTUATIONS IN SETTLING BASINS OF HEPS

Responsible for Research: I. I. Kukhianidze, Candidate of Technical Sciences, Senior Research Worker

Owing to the lack of reliable specifications and recommendations on the subject, field and laboratory investigations for the determination of the most efficient designs and location of screens for leveling out the velocity fluctuations in the settling basins of HEPs were carried out during 1957-58.

Field testing was carried out in the two operating settling basins, namely, those of the Bzhuzha and Samgorskaya HEPs. Laboratory experiments were conducted on a large-scale (1:6) river-channel type of model of the settling basin for the Bzhuzha HEP, and on a glass flume. Both field and laboratory tests consisted mainly in measuring and determining the flow-velocity distribution beyond the screens, in the different cross sections of the settling chambers of the basin. Low velocities were measured by current meters of the TNISGEI-Kh-6 and Kh-2 type, especially designed for the purpose.

The investigations revealed a nonuniform distribution of flow velocities in the basins, which points to poor design of the velocity-leveling screens made of angle bars and located in the transitional or approach sections of the basin chambers.

Laboratory experiments on screen bars at eight different cross sections yielded the following results with regard to the efficiency of design and location of screens:

1. For fluctuations in the mean approach velocities, varying from 0.6 to 7.7 m/sec, screen bars of rectangular, angular and round cross sections were found to ensure a more or less uniform velocity distribution in the chambers of the basin.

2. The most uniform velocity distribution in the basin chambers has been obtained with a double-row screen made of rectangular bars and situated in the chamber inlet. (The screen bars are arranged vertically in a checker-like pattern between the rows). With a screen-mesh factor of 0.600 to 0.670,

this type of screen should be made from bars 60-100 mm thick, with a width 1.5 to 2.0 times their thickness. The gap between the bars should be 1.5 to 2.0 times their thicknesses; while the distance between the rows should be equal to the gap between the bars.

3. Second best velocity distribution in the basin chambers is achieved with a screen made from angle bars, the screen-mesh factor varying between 0.400 and 0.600. The number of screen rows and the relative position between the bars and screen are the same as in the rectangular-bar screen, the width of angle bars should be 50 to 80 mm, the gap between the bars, 1.5 to 2.0 times the web width, and the distance between rows - 1.5 to 2.0 times the gap.

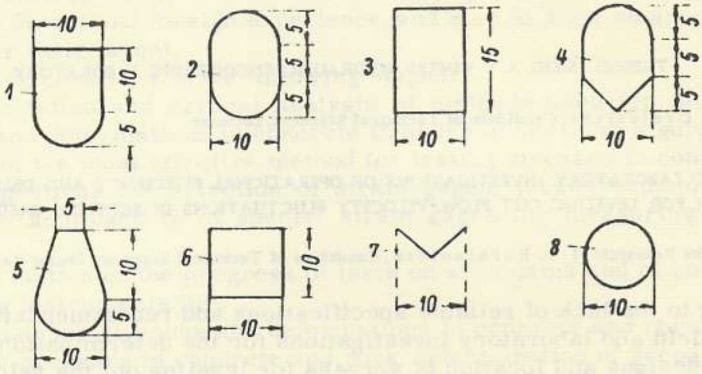


FIGURE 27. Cross section of bars

a–oval-rectangular; 2–oval; 3–rectangular; 4–oval-angular; 5–pearlike; 6–square; 7–angular; 8–round.

4. Third best results were obtained with a round-bar screen and a screen-mesh factor of 0.500-0.670. Arrangement of bars and screen is the same as above; bar diameter 50-75 mm; gaps between bars, 1.5-2.0 times their diameter; distance between rows, twice the bar diameter.

5. Loss of pressure in screens is found by equation

$$\Delta h = \zeta \frac{v^2}{2g},$$

where v = mean approach velocity in front of the screen;

ζ = coefficient of hydraulic resistance to be determined by empirical functions;

a) for screen bars of rectangular and round cross section

$$\zeta = \frac{m}{s} - n.$$

Experimental data for rectangular bars: m = 16.6 and n = 0.155; s = width of gap between bars; for round bars: m = 11.8 and n = 0.085;

b) for angular bars

$$\zeta = ms + n,$$

where m = - 0.096; n = 2.42; s = width of gap between bars.

The redesign of screens for operating basins according to the above recommendations will probably increase their efficiency, extend the service life of the water-intake structures and equipment and also reduce the operational expenses. In the design of new settling basins it will be possible (on the basis of the above data) to reduce the safety margin for the working length of the settling chambers and, hence, to reduce their construction cost.

TNISGEI IMENI A. V. VINTER HYDRAULIC ENGINEERING LABORATORY

Head: Professor Yu. Ya. Staerman, Doctor of Technical Sciences

INVESTIGATION OF THE GUMATSKAYA HEP DAM

Responsible for Research: G. I. Chilingarishvili, Candidate of Technical Sciences, Senior Research Worker

The Gumatskaya HEP-I concrete gravity dam on the Rioni River (Georgian S.S.R.) has an over-all height of 53 m and a height of the spillway section from the base to the crest of 42.5 m. The dam is divided by transverse temperature joints into 17.5-27.0 m long sections. Each section was made of individual blocks laid in tiers. The length of the blocks is equal to the above distance between the joints, the width is 6-8 m and the height 2.5 to 3 m.

Two galleries run inside the dam body along its longitudinal axis, the distance between the galleries and the upstream face being 3.5 m.

The foundation of the dam consists of fissured diabase rocks. There is a 15 m deep grout curtain at the upstream face of the dam. The average air temperature in the region of the dam, taken over a number of years, is +14.4° C.

During the erection of the dam, opening of construction joints and vertical cracks in some of the blocks of the lower tiers have been observed. This disturbance of the monolithic character of the structure results from thermal processes as a result of heat liberation in the concrete. Therefore, it was decided to investigate the thermal processes within the dam body and the nature of the resulting disturbance of the monolithic nature of the masonry.

A middle cross section of one of the sections of the spillway part of the dam has been chosen for the control of temperature fluctuations in the masonry. At the beginning of the field investigations, the section had been erected up to a height of 26 m. Three 110 mm diameter boreholes were drilled in this part of the massif through its whole thickness and electric-resistance thermometers were placed in pairs for the purpose of reciprocal checking, the distance between the check points being 3-5 m. This series of thermometers permitted observation of the continuous gradual cooling of the dam core until a constant temperature was reached.

In the upper part of the section, the electric-resistance thermometers were embedded in the blocks during concreting. The measurements showed that at an initial temperature of the concrete mix of 18-20°C and a cement content of 220-250 kg/m^3 (grade 400 pozzolana portland cement of the Kaspsk Cement Mill) the maximum increase in the temperature within the block took place 3-4 days after concreting and reached 20-24°C.

To investigate the nature of the disturbances in the monolithic structure of the masonry, five inclined boreholes, 110 mm in diameter, normal to the dam axis, were drilled from the lower dam gallery toward the tailwater side. The boreholes were spaced 4-10 m apart, at an angle of 67-73° to the vertical and were 13.1-23.3 m long. The purpose of these boreholes has been to intersect the assumed open vertical joints in the concrete structures and the cracks running along the dam axis. Separate hydraulic testing (in sections) of the boreholes and investigation of their walls with the help of optical instruments were carried out. They revealed in each 10-15 m, inside the masonry, spots of disturbance of the monolithic structure in the form of opened vertical joints or cracks. The joints and cracks extended throughout the whole length of the concreting blocks. Photographs of the 1.5-2 mm wide water-carrying cracks were obtained with the help of a special device [called in Soviet terminology - photoburoscope] designed by Engineer H. M. Viktorov for the taking of photographs through boreholes.

On the inside walls of the gallery the vertical joints between the blocks (owing to considerable length (up to 27 m) of the latter) were found to be loosened; the blocks were also found to slide horizontally against each other, the latter being due to the time lapses (up to 1.5 months) between the concreting of the superimposed tiers.

Tests of sealing quality and of strength of the concrete structures yielded positive results.

The results of these investigations made it possible to improve the monolithic character of the concrete masonry and to reduce to a minimum the volume of grouting work. Grouting was carried out before the dam was placed under head, i.e. after the core dam reached a constant temperature. For each linear meter of grouting boreholes an average of 100 m^3 of cement grouting has been carried out. After putting the dam into operation no seepage through the grouted masonry was observed.

The results of the investigations at the Gumatskaya HEP dam point once more to the fact that leaving the surfaces of a concrete block exposed to air for considerable intervals during concreting, in the hope that the exothermic heat will dissipate itself, fails to ensure a sufficient cooling of the concrete masonry. On the other hand, the long intervals between the laying of superimposed tiers of concrete lead to the appearance of sliding phenomena in the horizontal joints. In designing high concrete dams it is therefore necessary to provide for an artificial means for temperature control and to grout the joints between the concrete blocks after the temperature required for proper sealing of the joints is attained.

The checking of the temperature condition at the Gumatskaya dam is being continued. The experimental data thus obtained furnish new information on the laws governing heat dissipation within concrete blocks. On the basis of existing data on the operation of the Gumatskaya dam, conclusions were drawn on the suitability of dividing the dam in blocks and on the duration of cooling of the arch-dam body of the Ladzhanura HEP. Technical information

gathered during the results of investigations on the Gumatskaya dam may be used in selecting the means for temperature control and for ensuring the monolithic character of high-dam concrete structures.

APPROXIMATE VARIATIONAL-ROD METHOD FOR CALCULATION OF ARCH DAMS

Responsible for Research: K. M. Khuberyan, Doctor of Technical Sciences

In the rod method for calculation of arch dams, advanced by Kh. G. Ganev, the dam structure is divided into a number of cantilevers supported on a rigid foundation made of arches. Kh. Ganev suggests to solve the differential equation of the central cantilever supported on a rigid foundation, for each given case, by the method of successive approximation, whose basic idea is similar to that of the iterative method. Calculation of an arch dam may be still more simplified and less laborious, and the solution obtained more general, if the problem is solved by variational methods, for instance, the method developed by B. G. Galerkin.

The approximate equation of the deflected axis of an only slightly curved, rigidly embedded cantilever, may be written in the following way:

$$w = a_1 \varphi_1(y) + a_2 \varphi_2(y) + a_3 \varphi_3(y), \tag{1}$$

where

$$\left.\begin{aligned} \varphi_1(y) &= \left(1 - \frac{y}{H}\right)^2; \quad \varphi_2(y) = \left(1 - \frac{y}{H}\right)^2 \frac{y}{H}; \\ \varphi_3(y) &= \left(1 - \frac{y}{H}\right)^2 \left(\frac{y}{H}\right)^2; \end{aligned}\right\} \tag{2}$$

w = radial cantilever deflection;

y = ordinate of cantilever-axis point (axis of w passes through the free end of the cantilever toward the tailwater section; axis y passes downward through the cantilever embedment);

H = height of cantilever.

Applying B. G. Galerkin's method to the differential equation for a central cantilever, obtained by Kh. G. Ganev for the case when the influence of turning moments can be neglected, we obtain the following three equations:

$$\int_0^H \left[J(y) \frac{d^4 w}{dy^4} + 2 \frac{dJ(y)}{dy} \times \frac{d^3 w}{dy^3} + \frac{d^2 J(y)}{dy^2} \times \frac{d^2 w}{dy^2} + \right.$$
$$\left. + \frac{b e(y)}{r^2(y) \mu(y)} w - \frac{p(y)}{E} \right] \varphi_i(y)\, dy = 0; \; i = 1, 2, 3, \tag{3}$$

where $J(y)$ = moment of intertia of cantilever cross section;

$e(y)$ = wall thickness of cantilever;

$r(y)$ = radius of arch;

$p(y)$ = hydrostatic load per running meter or combined load per running meter from the water pressure and silt weight;

E = modulus of elasticity of the structural material of the dam;

b = width of cantilever (assumed to be equal to unit length);

$\mu(y)$ = dimensionless value which for each value of y is numerically equal to the radial deflection of a separate arch of constant cross section at its apex at the given elevation, the deflection resulting from a radial load uniformly distributed along the arch axis at $p = 1$, $r = 1$, $E = 1$, $e = 1$.

After introduction of expressions (1), (2) into the equations (3) and some additional transformations, the equations (3) may be represented in a canonic form.

Approximation of functions $e = e(y)$, $r = r(y)$, $\alpha = \alpha(y)$(α - half of the central angle of the arch) is done with the help of simplest, preferably algebraic functions. The expression for load $p(y)$ is then written. The function $\mu(y)$ is found with the help of the formula for the deflection of the apex of an individual arch element under a radial, uniformly distributed load and by means of functions $e(y)$, $r(y)$, $\alpha(y)$ (we choose a formula which corresponds to the boundary conditions for the dam under design). The integrals contained in the equations (3) brought to a canonic form are calculated and the equations are solved with respect to the parameters a_1, a_2, a_3. The part of the external radial load taken by the arches is found with the help of formula

$$p_a(y) = k_1(y)\, w(y), \tag{4}$$

where the coefficient

$$k_1(y) = \frac{Ee(y)}{r^2(y)\,\mu(y)}. \tag{5}$$

Each arch is calculated as an independently loaded unit (the load is assumed to be radial, uniformly distributed, and equal to $p_a(y)$). Each cantilever is also assumed to behave like an independent unit under the load $p(y) - p_a(y)$. This method may also be applied to the calculation of earthquake forces directed toward the headwater or tailwater side, if expression (5) included into the equations (3) be replaced by the coefficient $k_1(y)$ corresponding to the seismic load.

The method may be applied to the case when the deflection and the turning angle of the cross section of the central cantilever at its base are not equal to zero (instead of equation (1), (2), another equation is adopted, correponding to the given boundary conditions). The method is applicable to dams divided into horizontal tiers each made from different grades of concrete, i. e. having different moduli of elasticity. Considering the results obtained by Kh. G. Ganev, the method may be developed and extended for the case of thermal stresses distributed linearly through the body of the dam. For all the above forces and loads the method may also be extended for the case of acting turning moments.

For a more detailed computation of an arch dam in which the load is distributed between the arches and the cantilever, Kh. G. Ganev, assuming the displacements of the arches (in their keystone section) to be equal to those of several cantilevers, suggests a method in which the forces acting at the interface between a given cantilever and the adjacent arches depend upon the deflections of arches and the characteristics of the given cantilever,

but do not depend on the characteristics of the other cantilevers. However, in fact, the interacting forces depend upon the characteristics of the other cantilevers as well.

Conclusions

1. On the basis of the approximate method of calculating arch dams, which assumes the load to be distributed between arches and cantilevers, considering the displacements of the arches in their keystone (apex) section to be equal to those of a central cantilever and considering the load taken by the arch to be uniformly distributed, the efficiency has been proved of combining the scheme for calculating dams consisting of several cantilevers resting on an arch-type foundation (KH. G. Ganev's scheme) with the variational method of calculating cantilevers supported by plastic foundations. The variational-rod method for calculating arch dams, as proposed by the author, is less laborious, simpler, and provides for a more general solution to the above cantilever problem compared with Kh. G. Ganev's rod method.

2. Within the above assumption, the variational-rod method may easily be adapted to arch dams having different boundary conditions at the base of the central cantilever (a case not considered in this paper). It may also be extended to the case of thermal stresses linearly distributed through the body of a dam, as well as to the effect of shrinkage and swelling of concrete. The method may be made more accurate by taking into account the turning moments.

3. In order to extend the method of calculating arch dams in which the load is assumed to be distributed between the arches and the cantilevers (proceeding from the equality between displacements in arches and in cantilevers) further investigations are required to obtain a scientifically substantiated method based on the calculation scheme for several cantilevers on an elastic foundation.

SCIENTIFIC-RESEARCH DIVISION OF THE GIDROPROEKT IMENI S. Ya ZHUK
DEPARTMENT FOR INVESTIGATION OF HYDRO STRUCTURES

Head: R. N. Petrashen', Engineer

RESEARCH ON TRANSVERSE BENDING OF SLABS

Responsible for Research: N. D. Predtechenskii, Engineer

Research Team: A. K. Kuznetsova, Engineer
P. M. Vasil'ev, Engineer

A considerable number of elements of hydro structures behave as slabs subjected to different conditions of supporting and loading.

For simple cases, e. g. for slabs composed of beams, calculations can be effected resorting to the theory of strength of materials, whereas, for all

other cases, the theory of flexure of thin slabs must be applied. However, exact solutions obtained with the help of the thin-slab theory exist only for certain types of slabs operating under definite supporting and loading conditions, e.g. (elliptic plate embedded along its contour and subjected to a uniformly distributed load). Most of the other problems have to be solved by approximate or experimental methods.

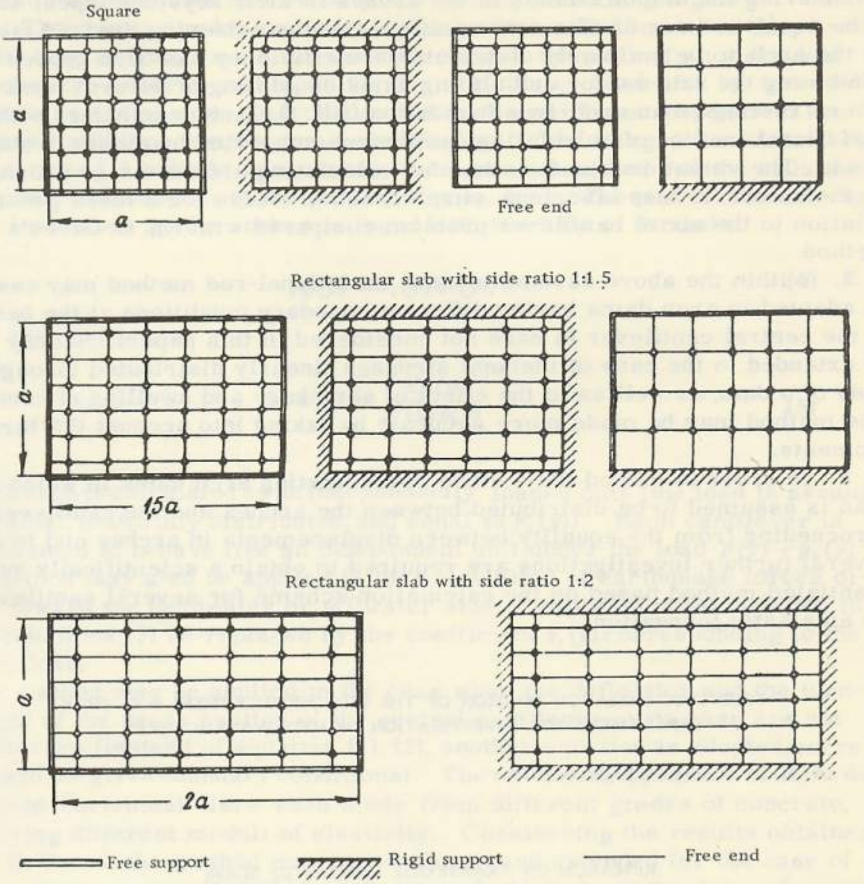


FIGURE 28. Scheme for the investigation of slabs, showing the points of application of single forces

All approximate methods require a considerable amount of calculation, especially for problems which deal with a partial loading, so that their practical use for design purposes is difficult. In this connection, special tables have been compiled by B. G. Galerkin, A. F. Smotrov, S. P. Timoshenko, Yu. A. Shimanskii and others, which are used in the design practice. These tables, however, can be applied only to a limited number of loading and supporting patterns usually met with in practice. Therefore, the above-mentioned research division carried out an investigation of rectangular slabs loaded by concentrated forces acting according to a given pattern. This investigation made it also possible to obtain solutions for a continuous load distributed according to any pattern whatsoever. The investigation has been carried out on a special electric simulating integrator of the EMBU-6 type. The slabs tested were simulated by a square lattice having the spacing: $h=\frac{a}{12}$ or $\frac{a}{8}$, where a is the short side of the slab. The calculated parameters were determined for the nodal points of the lattice.

The method for solving problems of transverse bending of slabs on the EMBU-6 electric integrator is described in the report published by the Research Division: "Electrical Solution of Problems of the Theory of Elasticity by Electric Simulation, 1957 (Reshenie zadach teorii uprugosti metodom elektromodelirovaniya)". Comparison with the existing solutions shows that the error in the solutions obtained by the described method is on the average 5% of its maximum [rated] value.

As a result of this investigation, reference tables for deflections, bending moments, torsion moments, shearing forces and support reactions have been issued.

The solutions were computed for the following types (Figure 28):

a) freely supported slabs along the contour, with side ratios 1:1, 1:1.5 and 1:2;

b) slabs, rigidly supported along the contour with side ratios as above;

c) square slabs with three sides freely supported and the fourth side free;

d) square slabs with one side rigidly fixed and the three remaining sides freely supported;

e) rectangular slabs with side ratio 1:1.5, with one long side rigidly fixed and the three remaining sides freely supported.

SOLUTION OF TWO-DIMENSIONAL PROBLEMS OF THE THEORY OF ELASTICITY FOR ELEMENTS SUBJECTED TO BODY FORCES

Responsible for Research: R. N. Petrashen', Engineer

Research by: K. I. Dzyuba, Engineer

Certain engineering problems require for their solution an examination of the two-dimensional stress distribution in elastic elements subjected to body forces. As is known, the solution of two-dimensional problems involving the interference of body forces is in general done by bringing the problem to boundary conditions described by a heterogeneous and – only in certain cases – a homogeneous biharmonic equation. A homogeneous equation

can be solved much faster than a heterogeneous equation. Thus by using the EMBU-6 electric simulating integrator, the computation may be accomplished in about half the usual time In the publications dealing with two-dimensional problems involving the action of body forces it is shown that a homogeneous biharmonic equation can only be applied for the case of action of constant body forces, or when the potential of the body forces represents a harmonic function.

This study deals with a series of cases in which the two-dimensional problem involving the action of body forces can be brought to a boundary problem described by a homogeneous biharmonic equation.

The paper gives a solution to the two-dimensional problem of the theory of elasticity for the following cases of distribution of the body forces X and Y:

a) the components of body forces on the coordinate axis are constant

$$X = \text{const.} \qquad Y = \text{const};$$

b) each component of the body force on the coordinate axis is a function of only one coordinate not corresponding to this component

$$X = X(y); \qquad Y = Y(x);$$

c) each component of the body force is a function of only one coordinate corresponding to this component

$$X = X(x); \qquad Y = Y(y);$$

d) the components of the body forces are equal to the sum of the two functions, each of which depends on only one variable

$$X = X(x) + X(y), \qquad Y = Y(x) + Y(y);$$

e) the components of the body forces are functions of two variables

$$X = X(x, y), \qquad Y = Y(x, y).$$

In addition, the paper deals with cases when the solution of a two-dimensional problem does not depend on the elasticity of materials.

The study has been used both for investigations on the EMBU-6 electric integrator of the state of stress in the Mamakan HEP massive buttressed dam subjected to the action of uplift, pressure, temperature, and tangential forces; and for the theoretical study of stress distribution in a buttress of variable thickness stressed by its dead weight.

INVESTIGATION OF THE PRESSURE ACTING ON THE BASE AND THE BACK FACE OF THE LOCK WALLS OF THE VOLGA HEP IMENI V. I. LENIN

Responsible for Research: I. Ya. Grodzenskaya, Engineer

Research by: M. A. Burmistrov, Engineer

Backfill pressure on the back face of the walls and the reaction of soil on the foundation of a structure depend on numerous factors such as: soil quality; type of structure; erection methods used, etc. Calculation of these factors is therefore rather difficult, and field measurement of the soil pressure on a structure would be of great practical value. It was for this purpose that an investigation of the lock walls of the Volga HEP has been carried out.

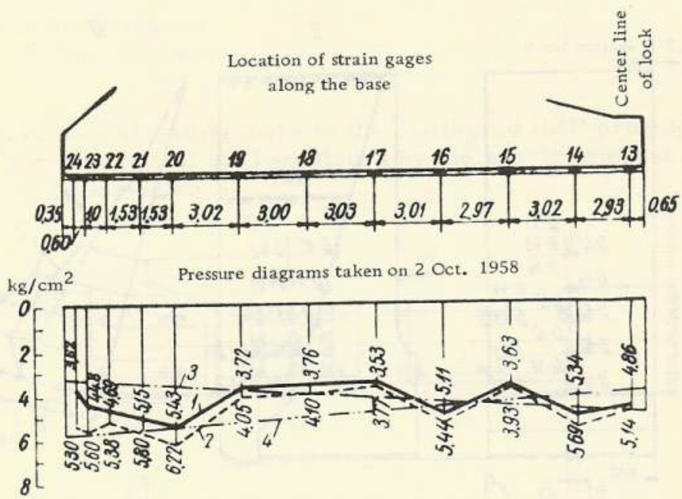


FIGURE 29. Pressure on the base of the sections (row No. 5)

a - pressure diagrams plotted according to the readings of the strain gages;
1 - at the level inside the lock chamber equal to the tailwater level;
2 - at the level inside the chamber equal to the headwater level;
b - graphs of rated pressure: 1 - at the level inside the lock chamber equal to the tailwater level; 2 - at the level inside the chamber equal to the headwater level.

The lock chamber is of the dock type with the floor sectioned transversally in the center. In the longitudinal direction the lock chamber is divided into sections 30 m long. The lock is built on thick sandy layers. The investigations were conducted on two sections of the chambers, Nos 5 and 7, using the remote-reading wire-type strain gages embedded during construction at three transversal rows in the back faces and in the base of the sections. In the back face of section No. 7 the gages were also located along three transversal rows, while in the section No. 5 the strain gages were installed in one row only, but in sets of three devices at each check point. In all, 106 gages were installed in both sections. The ground-water pressure was measured by piezometers installed into the base and the backfill.

Readings have been recorded since 1953 and cover the period of construction and two years of operation. Figure 29 shows the pressure diagram at the base of section No. 7 as taken in October 1958. Side-pressure diagrams for section No. 5 taken in September 1958 are presented in Figure 30.

The following conclusions could be drawn from the investigations:

During concreting and the resulting changes in load on the base, a time lag of 1 to 2 weeks has been noticed between the date of actual application of loads and their being recorded by the gages. Furthermore, during the temporary filling of the lock for ship passage, the pressure indicated by the gages amounted to 60-80% of the weight of overflow-water column. On the other hand, when emptying the lock in the winter (for repairs), the pressure indicated by the gages corresponded almost exactly to the weight of the overflow-water column.

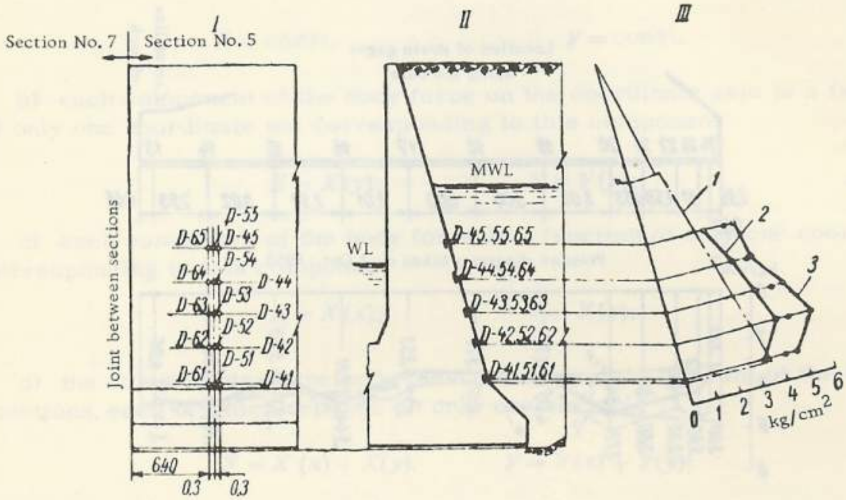


FIGURE 30. Soil pressure on the back face of the wall

I - back face of the wall; II - cross section of the lock wall; III - pressure diagrams taken on 30 September 1958; 1 - pressure calculated from Coulomb's formula; 2 - minimum recorded pressure; 3 - maximum recorded pressure. [MWL = maximum water level; WL = water level.]

The profile of the pressure diagrams in each transverse row, both in the base and the side walls, remained virtually unchanged since the beginning of construction work; only the magnitude of the coordinates changed in accordance with the increase in load.

With respect to magnitude and shape, the pressure graphs are close to the rated values, calculated with the help of formulas for nonuniform compression.

The diagrams of side pressure acting on the back faces considerably differ, both in shape and magnitude, from the theoretical graphs computed according to Coulomb's formula.

The total actual pressure on the back face was found to be by 25-30% higher than that calculated from the Coulomb formula. The intensity of pressure in the area of the entrance angle at the junction between the lock

walls and the floor is considerably lower than that recorded by the gages located at a higher level; this fact accounts for the curved profile of the graphs.

The temperature deformations of the lock walls and floors due to transition from winter to summer conditions and vice versa, both during construction and operation, are distinctly reflected in the pressure diagrams.

FIELD INVESTIGATION OF STRESSES IN THE TIE REINFORCEMENTS OF THE UPPER CHAMBER OF LOCK NO. 30 OF THE STALINGRAD HEP

Responsible for Research: I. Ya. Grodzenskaya, Engineer

Research Team: A. I. Tsarev, Engineer
V. M. Poduzov, Engineer

The design of the navigation locks of the Stalingrad HEP provides for prestressing the concrete of the lock floors by the gravity method developed by Professor A. Z. Bas'evich.

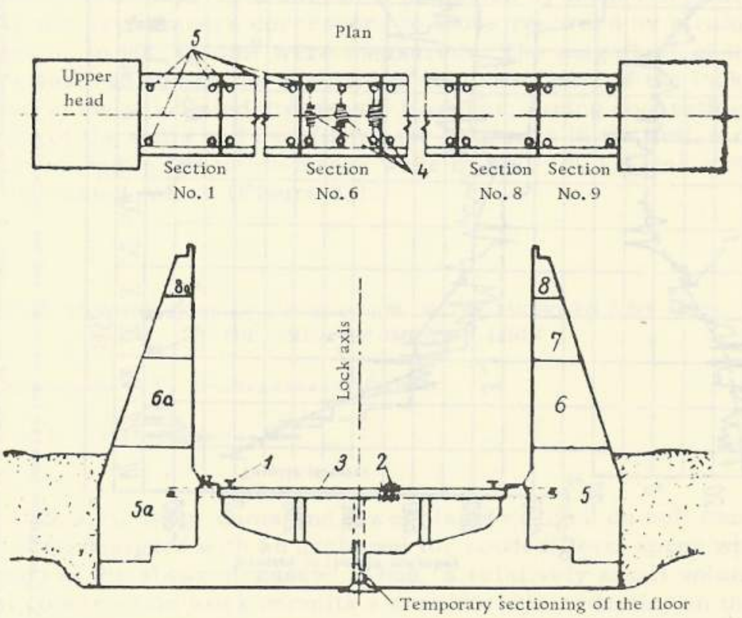


FIGURE 31. Location of measuring devices in the upper chamber of lock No. 30

1 - control mark for measuring elongation of the girders; 2 - strain gages for measuring stresses in reinforcements; 3 - tie reinforcement; 4 - strain gages for measuring stresses in reinforcements; 5 - control mark for measuring deformations in the ties.

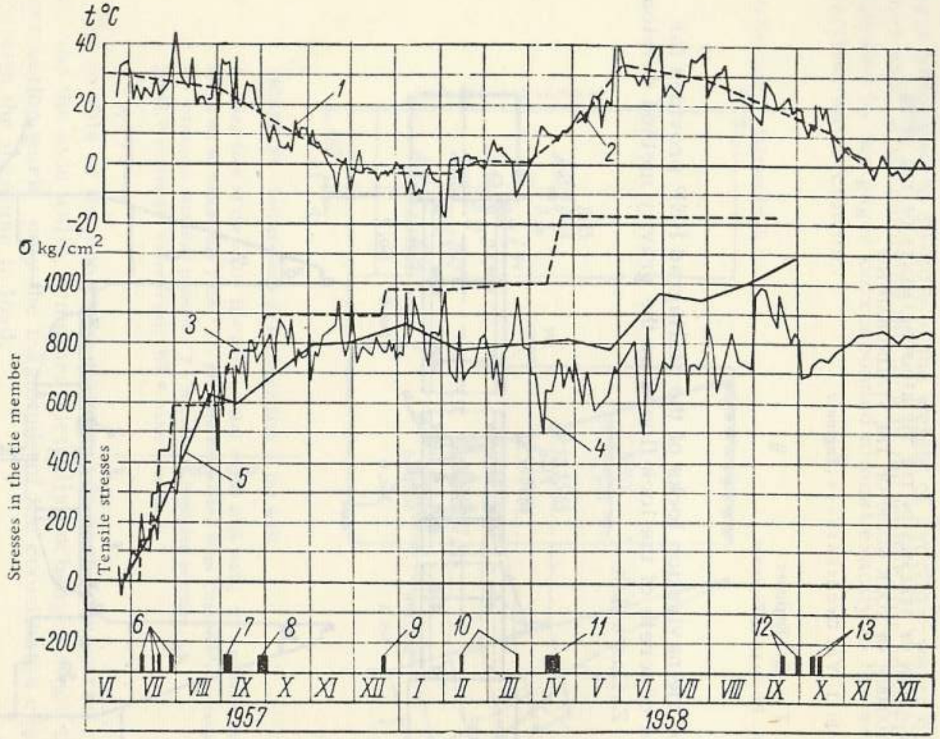


FIGURE 32. Comparison between the measured and the calculated stresses

1 - ambient temperatures at the time of measurement; 2 - mean ambient temperature; 3 - calculated stresses; 4 - stresses recorded by wire-type strain gages; 5 - stresses calculated on the basis of measured deformations; 3 - concreting of blocks; 7 - backfilling up to ∇- 7.5; 8 - backfilling up to ∇- 3.5; 9 - concreting of the block No. 63; 10 - backfilling from the left side up to ∇ 1.2, from the right side up to ∇ 0.8; 11 - concreting of blocks; 12 - concreting of the ceiling of water galleries; 13 - 3 backfilling from ∇ +3.0, from the left side up to ∇ + 7.0.

The prestressing of the concrete floor is achieved through the action of tie members which in turn are stressed due to the dead weight of the lock walls. The reinforcement of the ceiling of water galleries is used as a reinforcing element. The magnitude of stresses in the tie members depends both on the weight of walls and the extent of yielding of the foundation. The theoretical quantitive evaluation of the yielding of foundation is rather difficult.

Measurements of deformations and stresses appearing in the tie bars of the head of lock No. 30 have been undertaken with a view to correcting the sequence of concreting the chamber walls and the backfilling as well as to check the design assumptions.

The stresses were measured by wire-type strain gages in three reinforced girders of the tie member in one of the central sections of the lock. To ensure accurate and reliable measurements, the gages were arranged in groups of three.

The deformation in tie members were determined in all sections of the lock chamber by measuring the distance between the check marks situated at the ends of the tie members close to the lock walls. The location of the measuring gages is shown in Figure 31.

The results obtained in the course of the past one and a half years confirm the design assumptions concerning the behavior of a lock with a temporarily sectioned floor. The stresses computed by measuring the deformations in the tie members correspond to those recorded by strain gages.

Stresses up to 900 kg/cm^2 were measured. The magnitude of the stresses measured during the erection of the lower part of the lock walls correspond to the calculated stresses. However, during concreting of the upper parts of the walls and backfilling the recesses in the soil, stresses actually measured in the tie members were by 600-400 kg/cm^2 smaller than the calculated values (Figure 32).

FIELD INVESTIGATIONS OF THE BEHAVIOR OF THE ANCHORED FORE APRON AT THE VOLGA HEP IMENI V. I. LENIN

Responsible for Research: I. Ya. Grodzenskaya, Engineer

Research by: A. I. Tsarev, Engineer

In the U.S.S.R. large dams and power plants erected on soft foundations are very often designed with an anchored (or bonded) fore apron which carries part of the shear stresses. Thus, a relatively small volume of additional construction work permits a considerable reduction in the amount of concrete for the main structure and consequently a reduction of the total construction cost. Notwithstanding the large-scale application of anchored fore aprons, reliable information on their share in carrying the shear stresses is still lacking. It should be mentioned that a study of this problem became more and more urgent in connection with the introduction of lightweight structures and in view of the tendency to shift the main part of shear forces to the fore aprons. This means that the accepted requirements for independent stability of a structure are no longer valid.

In order to study the behavior of a fore apron, an investigation has been conducted at the Volga HEP which is erected on a thick layer of clayey soil. The fore apron of the power plant consists of a reinforced-concrete slab 0.75 m thick, waterproofed by a bituminous coating and covered by a layer of soil and several concrete slabs. The reinforcing bars of the fore apron, 80 mm in diameter, are attached at one end to the foundation of the powerhouse and, at the other, to the foundation of the thrash-retaining racks. Sections of the fore apron adjoining the powerhouse and the trash-retaining racks are made of asphalt concrete, thus allowing their independent settlement.

The investigation has been carried out on one section of the HEP by use of remote-reading wire-type strain gages. Eighteen gages installed on the reinforcing bars at six check points in the flexible section [of the apron] adjoining the powerhouse were sufficient to record the full load transferred to the fore apron. Apart from measuring the stresses in the reinforcement, the temperature in the bars and the extent of settlement were observed.

The gages are under continuous supervision since 1955, the date of their installation.

The recent inspection period covered the last stage of construction, filling up the reservoir, and a one-year operation period. The results of the study are given in Figure 33. The construction period is characterized by sharp changes in stresses in the reinforcement, a fact which reflects the diversity of acting factors. The absolute value of stresses depends upon the temperature prevailing during the closing up of the reinforcement [concreting]. During placing of the hot asphalt concrete and the increase in the ambient temperature, the reinforcement was found to have compressive stresses of various magnitudes (in certain bars these stresses reached 300-400 kg/cm^2). After the subsequent backfilling, the compressive stresses changed into tensile stresses averaging 360 kg/cm^2. Possible explanations of this change in the stresses are: temperature fluctuations; deformations in the foundation; horizontal thrust in the powerhouse and the trash-retaining structures; and possibility of local bending of the reinforcing bars as a result of heterogeneity of the foundation within the area of the flexible section [of the apron].

After the area had been flooded at the end of October 1955 and after the subsequent filling up of the storage reservoir, an almost identical rate of stress increase was observed in each reinforcing bar. The increase in stresses with the increase in the water head proceeds by stages. As can be seen in Figure 33, the increase of stresses in reinforcing bars shows good agreement with the diagram of calculated horizontal shear stresses acting on the structure. Between the periods of increase in stresses, a certain decrease in their value took place, the water head remaining practically unchanged. A sharp reduction in stresses occurred after the turbine units of section No. 8 were put into operation.

From the available data it can be concluded that the general stress pattern in the reinforcement of a fore apron points to the existence of a flexible connection of the fore apron with the powerhouse; this conclusion is supported by the fact that the stresses not only increase with the rise of the water head but also decrease with its drop. In the course of time a certain redistribution of stresses between the fore apron and the structure itself has also been noticed. Thus, during the spring of 1957, at the time of the first filling of the reservoir to the rated level, stresses in the fore apron represented 22% of the total shear stress acting on the structure, while at

the time of the second filling (spring 1958) only 17% of stresses occurred in the fore apron. Such a decrease is probably due to the relaxation of stresses, whereas the above-mentioned sharp reduction which took place during the start of the turbine unit is due to the vibration of the structure.

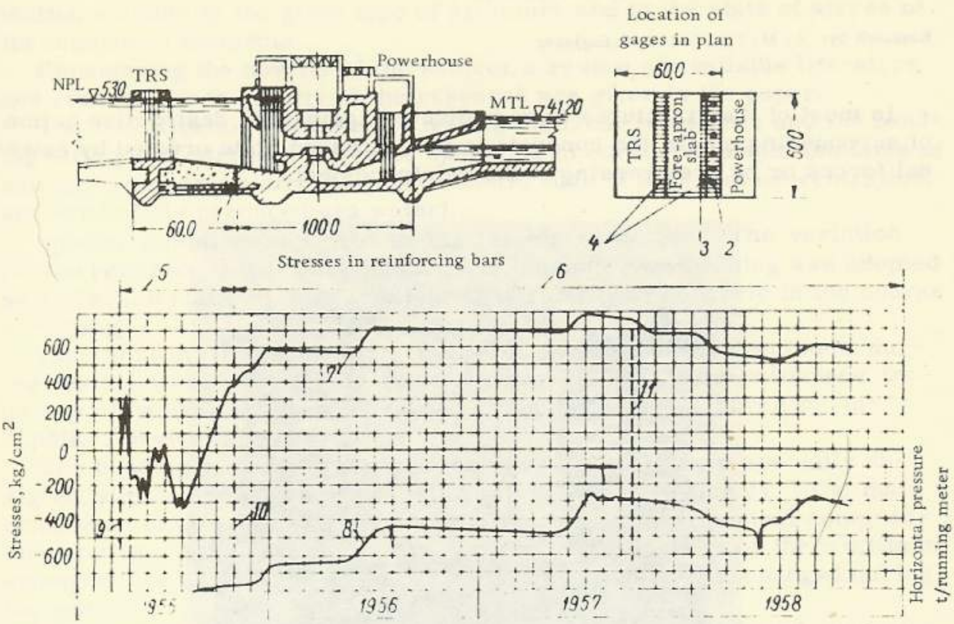


FIGURE 33. Stress pattern in the anchored fore apron of the Volga HEP imeni V. I. Lenin

1 - location of gages in the fore apron; 2 - location of gages; 3 - reinforcement of anchorage; 4 - flexible sections of fore apron; 5 - construction period; 6 - operational period; 7 - stresses in the reinforcing bars; 8 - horizontal forces on the powerhouse; 9 - date of installation of gages; 10 - flooding of excavation pit; 11 - date of starting of turbine operation.
[NPL = normal pondage level; TRS = trash-retaining structure; MTL = maximum tailwater level.]

NIIZhB, ASiA OF THE U. S. S. R. CENTRAL LABORATORY FOR CORROSION

Head: V. M. Medvedev, Candidate of Technical Sciences

EFFECT OF STRESSES IN CONCRETE ON ITS CORROSION RESISTANCE

Responsible for Research: Professor V. M. Moskvin, Doctor of Technical Sciences, Associate Member of the ASiA of the U. S. S. R.

Research by: A. M. Podval'nyi, Engineer

In most of the structures to be protected against the destructive action of surrounding media, the concrete is in a stressed state created by external forces or by prestressing of the reinforcement.

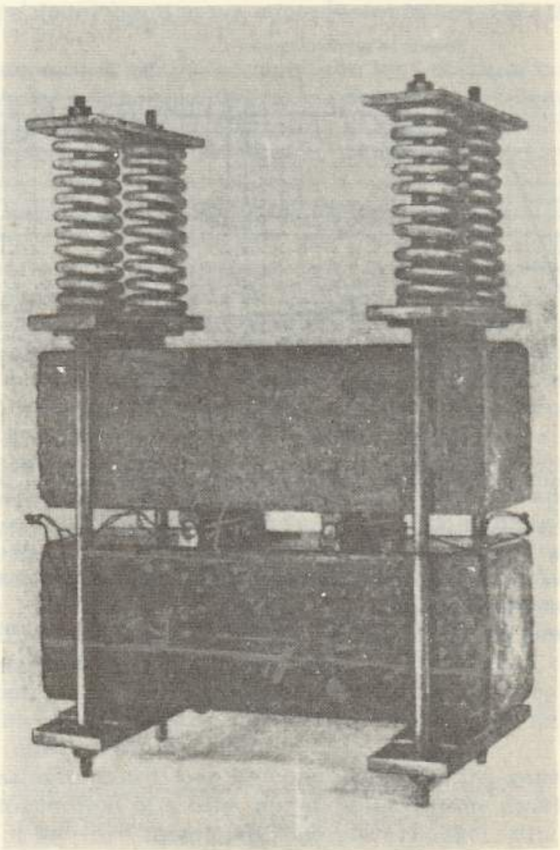


FIGURE 34. Stressing unit with specimens

The conditions for laboratory testing of corrosion resistance of concrete differ from the actual working conditions of concrete structures in that concrete is tested in the nonstressed state. It is therefore difficult to select suitable concrete mixes and protective means, and to apply the conclusions reached from the experiments to the construction practice.

The basic aim of this investigation has been to determine experimentally the stresses in a concrete structure, with a view to applying corrosion protection suitable to the given type of structure and to the state of stress of its component elements.

Considering the novelty of the subject, a review of available literature and reasons for the utility of the research are given in the paper.

The investigation consisted in testing the frost resistance and the bending strength of stressed concrete operating in corrosive media (sulfates of natrium, magnesium and calcium; caustic soda of different concentration; and artificially produced sea water).

Special attention was paid to the testing technique. The variation in concrete strength at pure tension and tension-upon-bending was adopted as a characteristic of the variations in durability of concrete in the course of testing.

Variations in strength and internal structure of the test specimens during testing were determined from the variations in the resonance frequency of natural oscillations caused by bending, as measured by the IChMK-2 device.

The bending stresses in specimens were produced by a specially designed, simple and reliable stressing unit shown in Figure 34. The main component of this unit is a preliminarily calibrated cylindrical compression spring. The specimens were stressed to a fraction of their ultimate strength; the numerical value of the stresses remained unchanged during the test.

The study proved that the joint action of tensile stresses and corrosive media produces greater destruction of concrete than would result from the simple superposition of either effect. These results have been obtained for all tested corrosive solutions and different concentrations. In most cases compressive stresses are capable of retarding the destructive process, not only in stretched (tensile-stressed) specimens, but also in nonstressed specimens.

The frost resistance of concrete (considered here as a particular case of its over-all durability) was found to depend also on the type and intensity of stresses in a concrete specimen or structure.

Figure 35 represents the diagram of the dynamic modulus of elasticity for test specimens (concrete beams 7X7X22 cm) frozen in the stressed state after bending, and thawed in water and in a 5% solution of natrium sulfate.

The frost resistance of a concrete stressed in tension was found to be lower than that of a compressed or nonstressed concrete.

The experiments proved that the destructive effect of bending, freezing and thawing of concrete, kept in corrosive media, is not additive.

It was found that the fracture surface and the type of surface defects in specimens frozen in the stressed state differ markedly from that of specimens frozen in the nonstressed state.

Water absorption of stressed specimens during the initial stage of testing was also found to be higher than that of nonstressed specimens.

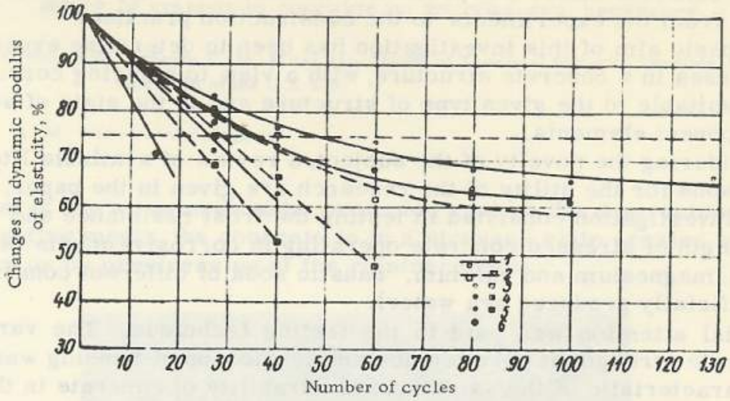


FIGURE 35. Frost resistance of stressed-concrete specimens

1 - thawing in water; 2 - thawing in 5 % Na_2SO_4; 3 - nonstressed specimens; 4 - specimens stressed to 0.2σ; 5 - specimens stressed to 0.35σ; 6 - specimens stressed to 0.5σ.

Durability tests for stressed concrete represent a new, highly valuable research tool, as it makes it possible to reveal certain peculiarities and quantitative characteristics concerning the mechanism of destruction of concrete upon freezing.

Observations showed that under field conditions destruction of concrete also starts in the most stressed extended sections of a structure. However, no particular attention has been paid to this fact, and information available on this subject is very scarce. There are enough reasons to believe that field tests of concrete structures located in corrosive media should be performed, taking into consideration the constructional features and the state of stress of a structure.

A working hypothesis is advanced, based on test results and certain theoretical considerations and literature references, explaining the mechanism of the effect of stresses on the corrosion resistance of concrete. This hypothesis proceeds from certain characteristic features of the internal structure of concrete, from considerations concerning the static fatigue in concrete under the action of tensile stresses, and from the role of corrosive media in the development of cracks.

The authors of the study hold that the strength of a stressed concrete depends ultimately on the presence of a structural component unstable in the given medium and susceptible to destruction under its effect.

The great number of possible combinations of durability and strength factors in concrete and in the hardened cement stone is responsible for considerable variations in the durability of concrete as observed in practice. The most disadvantageous combination results when the same structural component has both lowest strength and lowest durability in the given medium.

The conclusions reached at the investigation permit the understanding of certain aspects of failure of concrete structures in corrosive media. Objectives for further investigations were set with a view to developing design and constructional solutions which would ensure the durability of concrete structures.

NIIZhB, ASiA OF THE U.S.S.R. LABORATORY FOR THE STUDY OF REINFORCEMENTS AND REINFORCED CONCRETE

Head: Professor A. A. Gvozdev, Doctor of Technical Sciences; Member of the ASiA of the U.S.S.R.

TEMPERATURE AND SHRINKAGE STRESSES IN PLAIN CONCRETE AND REINFORCED-CONCRETE STRUCTURES

Responsible for Research: S. V. Aleksandrovskii, Candidate of Technical Sciences, Senior Research Worker

This is a report on extensive experimental investigations of some aspects of shrinkage, swelling and temperature stresses in concrete.

The following problems have been studied: the effect of prolonged external loading on drying and shrinkage of concrete; the relationship between the deformation in concrete and the fluctuation in moisture content upon drying and wetting; hysteresis in concrete deformation caused by repeated drying and wetting cycles; physical properties of concrete as related to temperature and shrinkage stresses due to the heat and moisture exchange in the concrete.

The following conclusions have been drawn:

1. Prolonged external loading under normal conditions accelerates the drying-out of concrete, with the possible exception of very thin elements with high surface modulus, in which the drying-out may be delayed under the effect of compressive loads.

2. The higher the stresses and the lower the strength of concrete, the greater the effect of external prolonged loading on the drying conditions.

3. Shrinkage and creep deformations are not additive. Full deformations in a loaded noncoated test specimen differ from the sum of the shrinkage and creep deformations in identical specimens: one – noncoated and unloaded, and the second – loaded and coated.

4. Shrinkage of a compressed specimen is higher, whereas that of a specimen stressed in tension is lower than the shrinkage of the unloaded [reference] specimen. The lower the strength of concrete, i.e. the higher the W/C ratio and the lower the concrete age, and the higher the stresses, the more pronounced is the above difference.

5. The generally accepted technique for investigating creep in concrete does not take into account shrinkage deformations. Testing results for unloaded [reference] specimens show that the creep values for specimens subjected to compressive stresses are exaggerated, whereas those for tensile-loaded specimens are underrated. Creep tests should therefore be carried out without taking into account shrinkage deformations by using coated specimens protected from drying-out.

6. There is no definite relationship between shrinkage in concrete and the reduction in moisture content upon drying-out or between swelling and the increase in moisture while wetting the concrete. Drying-out of concrete to the critical moisture content does not cause shrinkage, which develops only upon reduction of the moisture content to below that of the critical point. Likewise, wetting of concrete to a point below the critical moisture content would not be sufficient to produce swelling, which develops only upon increase in the moisture content to above that of the critical point. This indefiniteness of the above relationship is especially noticeable during the shrinkage of drying concrete and less distinct during its swelling upon wetting.

7. The amount of relative critical moisture during shrinkage depends essentially on the cement content of the mix, and increases with the increase of the latter. At a conventional cement content $C(kg/m^3)$ this value may be determined by the formula

$$U_{cr}^{shr} = 3.7 \cdot C \cdot 10^{-5} \ (g/g).$$

8. The value of the relative critical moisture content during swelling depends essentially on W/C ratio and increases with the increase of the latter. At a conventional W/C ratio this value may be found from the formula

$$U_{cr}^{sw} = 64\,(1 + 9.2W/C) \cdot 10^{-5}\ (g/g).$$

9. The temperature coefficient of concrete shrinkage is much higher than that generally accepted (15°C). Coefficients exceeding this number four to five times have been obtained by tests.

10. The physical pattern of swelling differs from that of shrinkage: curves of swelling and shrinkage differ in shape.

11. Shrinkage and swelling deformations are irreversible. At repeated alternate drying and wetting of concrete, hysteresis of deformations is clearly observed.

In the course of time, with aging of concrete, the hysteresis loop is gradually flattened and stabilized. Absolute limiting values for shrinkage and swelling deformations and those for variations in relative moisture content decrease with time.

12. Volume changes in the gel component of the hardened cement stone have a predominant effect on deformations in concrete resulting from fluctuations in its moisture content. This conclusion conforms with recent concepts on the nature of the shrinkage process, as expressed in the theory developed by Professor A. E. Sheikin.

13. A low-setting concrete with a low W/C ratio and low cement content is preferable since it does not yield undesirable thermal-shrinkage stresses.

14. The coefficient of moisture diffusion K and of moisture losses β from an open surface depend on the W/C ratio and the amount of cement $C(kg/m^3)$. For a conventional W/C ratio and C, these coefficients may be determined from formulas

$$K = 6\left(1 - 0.2\,\frac{W}{C}\right) \cdot \left(1 + \frac{C-300}{375}\right) \cdot 10^{-6}\,(m^2/hr);$$

$$\beta = \left(3.3 \cdot \frac{W}{C} - 1\right) \cdot \left(1 + \frac{C-300}{200}\right) \cdot 10^{-4}\ (m/hr).$$

15. The coefficient of linear shrinkage β for concretes with the same cement content does not depend on the W/C ratio, composition or age of concrete. For conventional concrete $\beta = 3.5\times10^{-6}\left(\frac{mm/mm}{g/g}\right)$.

16. The coefficient of linear swelling η which is the counterpart of the coefficient β is variable and depends on the W/C ratio.

For concrete with a conventional W/C ratio, η may be determined from the formula

$$\eta = \left(1{,}8 - 1{,}4\,\frac{W}{C}\right)\cdot 10^{-6}\left(\frac{mm/mm}{g/g}\right).$$

NIIZhB, ASiA OF THE U. S. S. R. LABORATORY FOR TECHNOLOGY OF MANUFACTURING PRESTRESSED REINFORCED-CONCRETE STRUCTURES

Scientific Guidance: NIS Orgenergostroi

INVESTIGATION AND DEVELOPMENT OF CONSTRUCTION METHODS FOR THE USE OF PRESTRESSED PRECAST MONOLITHIC STRUCTURAL ELEMENTS

Responsible for Research: Professor V. V. Mikhailov, Doctor of Technical Sciences, Member of ASiA of the U. S. S. R.

Research by: O. V. Mikhailov, Engineer

Reinforced-concrete structures produced from prestressed reinforced-concrete elements for industrial, civil and transportation engineering are being increasingly used both in the U. S. S. R. and abroad.

However, large-scale introduction of the so-called precast monolithic elements into the building of hydro structures has been so far hampered by insufficient knowledge of their behavior and by the lack of methods for calculating strength, resistance to cracking, and rigidity.

Recent investigations carried out by different research institutes established that this type of structural elements has certain specific features and occupies an intermediate position between conventional reinforced-concrete structures and prestressed structures.

In dependence on the amount of prestressed elements used in the structures, on the degree of prestressing, etc.,the properties of precast monolithic structures may be adjusted at will and brought to the level of those of conventional prestressed structures in which the entire cross section of concrete is prestressed.

The investigations, carried out between the years of 1956 and 1958, dealt with the study of properties of precast monolithic structures, and were effected on large-size specimens of beam and slab shapes (volumes of 1.7 to 4.0 m^3) of rectangular cross section. Some specimens were reinforced with prestressed T-shaped reinforcing bars, not only in the tension but also in the compression zones.

The following conclusions have been drawn from this study:

1) minimum extensibility * of a monolithic nonstressed concrete is 30×10^{-5}. The extensibility may be increased up to $(45\text{-}60) \times 10^{-5}$ by fluting the surface and the externalization of stresses in the reinforcing elements;

2) at the instant of crack formation in the prestressed elements, the width of the cracks in the monolithic concrete does not exceed the permissible value of 0.2 mm;

3) cracks in monolithic concrete do not propagate so rapidly as in conventional reinforced-concrete structures;

4) in precast monolithic structures with the prestressed elements reliably fastened to the monolithic concrete (three-side contact by means of elements with rectangular cross section; full-surface contact by means of T-shaped elements; T-shaped overlapping projection of reinforcing bars, etc.) deformations occur simultaneously in the monolithic concrete block and in the prestressed elements located both in the compression and in the tension zones, the magnitude of these deformations being at times so high as to cause even destruction (failure) of the structure;

5) rigidity of precast monolithic structures is identical with that of prestressed structures;

6) the failure of a precast monolithic structure reinforced with prestressed elements develops according to the classic scheme of a flexible monolithic reinforced-concrete structure. Upon completion of the investigations a report has been prepared which included:

a) a new interpretation of the theory of prestressed elements and particularly of precast monolithic structures. These new ideas were underlying the specifications developed by Orgenergostroi jointly with the Scientific Research Institute for plain and reinforced concrete of the AS and A of the U.S.S.R. and termed: "Regulations for the Design and Manufacture of Precast Monolithic Elements for Hydro Structures";

b) methods for calculating strength and crack resistance of monolithic precast structures according to their limiting state of stress;

c) analysis of calculation formulas; the analysis confirmed the marked effect of the magnitude of extensibility of a monolithic concrete on the size of the cross section of the prestressed elements and, in general, on the load-carrying capacity of a precast monolithic structure at the very moment of crack formation in a monolithic prestressed concrete.

Up to date there are no regulations or instructions whatsoever dealing with the calculation and design of precast monolithic hydro structures reinforced with prestressed elements.

Therefore, the regulations drawn up by Orgenergostroi jointly with the Scientific Research Institute of the AS and A of the U.S.S.R. permit the design and building agencies to widen the scope of this modern and economical type of structural element.

The regulations for design and construction of precast monolithic hydro structures supplement the existing "Regulations for the Design of Prestressed Reinforced-concrete Structures (SN-10-57)".

A detailed study of technical and economical aspects of precast monolithic structures and their comparison with conventional monolithic reinforced concrete has been carried out by Mosgidep. This study was based

* [The capacity of the reinforcing member to be elongated.]

on data on the Kaunas HEP which has been designed for both alternatives: precast monolithic and conventional reinforced concrete.

It was shown that the use of the precast monolithic structural elements results in considerable savings in concrete – 15-25%; in reinforcing steel – 150-250%; in consumption of cement for the mix – 15-25%. Calculated savings in labor expenditure for excavation work represent 30%.

Comparison of the estimated cost of both alternatives showed that precast monolithic structures are by 20% less expensive. The most efficient use of precast monolithic structural elements in hydro construction would be for: retaining walls; lock chambers; dams and other structural elements of power plants like crane beams for bridge cranes, intermediate floors, foundations, etc.

MISI IMENI V. V. KUIBYSHEV CHAIR OF HYDRO STRUCTURES

Head: Professor M. M. Grishin, Doctor of Technical Sciences, Member of the ASiA of the U. S. S. R.

INVESTIGATION OF TEMPERATURE AND SHRINKAGE STRESSES IN THE CONCRETE BLOCKS OF HYDRO STRUCTURES PLACED IN WINTER

Scientific Guidance: Professor M. M. Grishin, Doctor of Technical Sciences

Responsible for Research: V. G. Orekhov, Candidate of Technical Sciences, Junior Lecturer

Research Team: B. I. Komzin, Postgraduate
A. I. Medovikov, Postgraduate
E. M. Romanov, Engineer
A. I. Churakov, Engineer

In accordance with the research schedule, the investigation has been conducted in several stages:

In 1956 the research team designed and constructed control and measuring equipment intended for checking the temperature inside a concrete block. During the winter 1956-57 a field investigation of the thermal conditions of concreting the blocks has been carried out. As a result of this study the parameters of thermal effects and the thermal-engineering properties of concrete have been established and model tests of the thermal behavior of blocks have been started.

In 1957 the team carried out extensive investigations of the thermal conditions in concrete blocks with the help of Professor V. S. Luk'yanov's hydraulical integrator (simulator) and analyzed the temperature-stress pattern in blocks. In 1957 the same chair designed instruments for measuring stresses in concrete blocks, which permitted the team to carry out field investigations of the temperature-stress pattern in concrete blocks. Theoretical and experimental studies furnished the basic information for the development of practical measures intended to ensure the proper conditions for concrete placing during the winter.

IGiG OF THE ACADEMY OF SCIENCES OF THE Ukr. S. S. R. HYDROTECHNICAL DEPARTMENT

Head: Professor B. A. Pyshkin, Doctor of Technical Sciences, Associate Member of the Academy of Sciences of the Ukr. S. S. R.

ELASTIC-THEORY PROBLEM FOR BODIES CHANGING THEIR SHAPE WHILE BEING LOADED (CONTRIBUTION TO THE THEORY OF ERECTION OF STRUCTURES)

Responsible for Research: L. I. Dyatlovitskii, Candidate of Technical Sciences

Research Team: L. I. Dyatlovitskii
L. B. Rabinovich

In the computation of stresses in massive structures all loads on the structure including those of the dead weight are assumed to act simultaneously and instantaneously after the structure has acquired its final form.

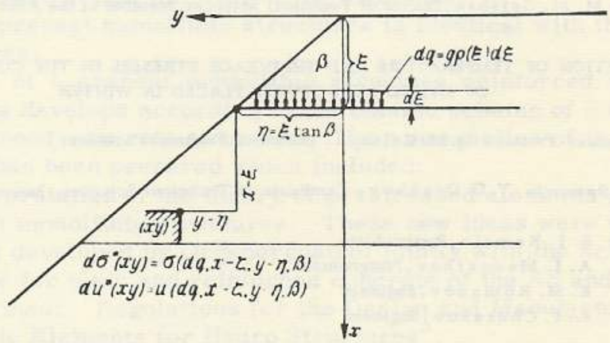


FIGURE 36. Infinite obtuse-angled wedge erected by elementary horizontal layers under conditions of a gravity field

In reality, during erection, simultaneous with the gradual application of load, changes are taking place in the shape of the structure being erected. The state of stress changes continually until the structure is completed, and its final pattern differs from that which forms upon the instantaneous application of loads.

Therefore, contrary to the conventional treatment of an elastic-theory problem in which the load is applied to a body having fully determined boundaries, a detailed investigation was undertaken of stresses in bodies whose contour changes during loading.

The solution to the elastic problem in this new formulation has been obtained in a finite form for two most simple cases:

1) for an infinite obtuse-angled wedge (Figure 36) erected in elementary horizontal layers under conditions of a gravity field (solution by E. I. Rashba);

2) round pipe (Figure 37) with its wall thickness continuously increasing to make up for the increase in the pressure on the internal surface of the pipe (solution by L. I. Dyatlovitskii).

A rigorously theoretical study of the problems with their formulation in the simplest way permits this calculation method to be applied in tentative form to more complicated problems met in practice. In such a formulation all stresses, dislocations and deformations are integrals of the same stresses that appear in a body of invariable shape under the action of elementary load increments.

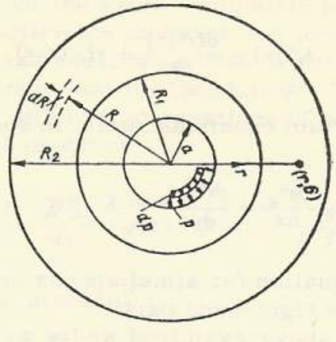


FIGURE 37. Round pipe with wall thickness continuously increasing to make up for increase in pressure on the internal surface of the pipe walls

For any given point, originated in the course of the build-up of a body, integration should be done only within the boundaries of an area which was formed after this point was originated. Hence, one of the integration limits is variable. For instance, the stresses, dislocations and deformations at a certain point (x, y) of a wedge upon completion of the processes of building up by layers having thickness $d\zeta$ are

$$\sigma^*(x, y) = \varkappa \int_x^0 \sigma(x, y, \zeta)\, d\zeta; \quad U^*(x, y) = \varkappa \int_x^0 U(x, y, \zeta)\, d\zeta; \; \varepsilon^*(x, y) = \\ = \varkappa \int_x^0 \varepsilon(x, y, \zeta)\, d\zeta, \tag{1}$$

where $\sigma(x, y, \zeta)$, $U(x, y, \zeta)$, $\varepsilon(x, y, \zeta)$ = stresses, dislocations, and deformations produced in a wedge with an invariable upper (horizontal) boundary $x = \zeta$, under a load q applied along the border $\varkappa = \frac{\rho_g}{q}$ – dimensional coefficient $\left[\frac{1}{l}\right]$.

Similarly we find for a pipe $\sigma^*(r) = \varkappa \int_r^{R_1} \sigma(r, R)\, dR$ etc.

In this case, the deformations are connected with the dislocations not by means of a conventional relationship of the type $\varepsilon = \frac{\partial U}{\partial x}$ etc. but by the following expression:

$$\left.\begin{aligned}\varepsilon_x^*(x, y) &= \frac{\partial u^*(xy)}{\partial x} + \varkappa\, U(x, y, x),\\ \varepsilon_y^*(x,y) &= \frac{\partial V^*(x,y)}{\partial y},\\ \gamma_{xy}^*(x, y) &= \frac{\partial U^*(x, y)}{\partial y} + \frac{\partial V^*(x,y)}{\partial x} + \varkappa V(x, y, x).\end{aligned}\right\} \text{For wedge} \qquad (2)$$

For pipe
$$\varepsilon_r^*(r) = \frac{\partial U^*(r)}{\partial r} + \varkappa U\,(r, r). \qquad (2')$$

The differential equilibrium equations retain in such bodies their usual form

$$\frac{\partial \sigma^*_x}{\partial x} + \frac{\partial \tau^*_{xy}}{\partial y} = -\,\mathrm{X} \quad \text{etc.} \qquad (3)$$

As a result of (2), the equation for simultaneous occurrence of deformations acquires the appropriate right-hand part.

For instance, for the above-examined wedge we have

$$\triangle^2\,[\sigma_x^*(x, y) + \sigma_y^*(x, y)] = -\,\varkappa\,\frac{\partial}{\partial x}\,\sigma_y(x, y, \zeta)\,|_{\zeta=x}\,. \qquad (4)$$

For pipe

$$\left(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\,\frac{\partial}{\partial r}\right)[\sigma_r^*(r) + \sigma_\theta^*(r)] = -\,\varkappa\left\{\frac{1}{r}\,\sigma_\theta(r, r) + \frac{\partial}{\partial r}\,[\sigma_r(r, R) + \sigma_\theta]\,\Big|_{R=r} + \frac{d}{dr}\,\sigma_\theta(r, r)\right\}. \qquad (4')$$

For intricately shaped bodies it is possible to obtain an approximate solution to the problem by dividing the erection process into a number of separate stages.

A solution in a finite form has also been obtained for bodies of arbitrary shape, by simplifying the problem and using the hypothesis of two-dimensional cross sections.

INVESTIGATION OF STRESSES IN SLABS RESTING ON AN ELASTIC FOUNDATION WITH ALLOWANCE FOR FRICTION FORCES AT THE CONTACT SURFACES (TWO-DIMENSIONAL AND AXIAL-SYMMETRIC PROBLEMS)

Responsible for Research: B. A. Pyshkin, Associate Member of the Academy of Sciences of the Ukr. S. S. R.

Research by: V. M. Seimov, Postgraduate

The existing methods for calculating slabs resting on elastic foundations do not take into account contact friction. The solution worked out by

L. I. Dyatlovitskii on the basis of V. A. Florin's method does not cover all the aspects of the problem since it does not allow for elongation of the strip and requires, moreover, a considerable amount of calculation work. I. I. Gudushauri's study is limited to the case of uniformly distributed loads and contains, moreover, an error. The present work is based on P. I. Klubin's method and takes into account contact friction. The normal $p(x)$ and tangential $\tau(x)$ reactions of a foundation have been represented by series from Chebyshev polynomes (for the two-dimensional problem) and Chebyshev and Legendre polynomes (for the axial-symmetric problem) divided by $\sqrt{1-x^2}$. The indeterminate coefficients included into these series have been found from the equilibrium and equality conditions for vertical and horizontal displacements of the slab base and the foundation. The vertical displacements of a slab are found by differential equations of bending:

1) two-dimensional problem

$$\frac{D}{l^4}\,\frac{d^4 w}{dx^4} = q(x) - p(x) - \frac{1}{2}\,\frac{h}{l}\,\frac{d\tau}{dx}, \tag{1}$$

2) axial-symmetric problem

$$\frac{D}{a^4}\,\frac{1}{r}\,\frac{d}{dr}\left\{\frac{d}{dr}\left[\frac{1}{r}\,\frac{d}{dr}\left(r\frac{dw}{dr}\right)\right]\right\} = q(r) - p(r) - \frac{1}{2}\,\frac{h}{a}\left[\frac{\tau(r)}{r} + \frac{d\tau}{dr}\right], \tag{2}$$

where h and l = height and half length of slab; a = slab radius; x and r = =relative coordinates.

The horizontal displacements of a slab consist of the displacement caused by bending (product of deflection angle and the half of height) and those occurring upon compression (or tension) as a result of reactions $\tau(x)$.

In computing compressive displacements within the framework of the axial-symmetric problem, a two-dimensional stressed condition produced by radial forces τr distributed through the median plane of the slab has been considered. With the introduction of the stress function related to the stresses by

$$f = r\sigma_r, \quad \sigma_\theta = \frac{df}{dr} - \frac{1}{h}\,r\tau(r), \tag{3}$$

the computation of compressive displacements reduces to finding the integral

$$f = c_1 r + c_2\frac{1}{r} + \frac{1}{h}\,\frac{1}{r}\int r\left\{\iint\left[(2+\beta)\,\tau(r) + r\frac{d\tau}{dr}\right]dr\right\}dr. \tag{4}$$

Vertical and horizontal displacements of the foundation surface upon loadings $p(x)$ and $\tau(x)$, due to loads $p(r)$ and $\tau(r)$, are found by integrating on a segment or a surface of a circle, the expressions for displacements caused by concentrated forces. Displacements of foundation excepting the horizontal displacement caused by $\tau(r)$ may also be determined by means of the Chebyshev and Legendre polynomes and their derivatives.

If the slab is loaded by partially distributed or locally concentrated forces (moments), the equations (1) and (2) should be integrated for two sections (areas).

The following types of loads have been considered in the work.

1. For the two-dimensional problem: load distributed uniformly on the whole slab or on a part of it; two vertical and horizontal symmetric and antisymmetric concentrated forces acting at an arbitrary distance from the slab center; two symmetric and antisymmetric moments at an arbitrary distance; antisymmetric linearly distributed load.

2. For the axial-symmetric problem: load uniformly distributed over the whole slab or over part of it; load distributed over the circumference of a circle of arbitrary radius; load concentrated in the center.

For the two-dimensional problems additional studies have been made: on the effect of lateral supplementary loads of the uniform and triangular, symmetric and antisymmetric type.

Stresses in the foundation (modeled in plexiglass) of a finite-rigidity strip (modeled as a metal beam) loaded centrally by a concentrated force have been determined experimentally by the photoelasticity method. The value of stresses σ_y and τ_{xy} were measured at the depth 0.1 L and 0.2 L (L-length of the beam); they satisfactorily agree with the results obtained by calculations.

Similarity criteria for bending, overturning angles, curvatures, and stresses in slabs were obtained by proceeding from the laws of modeling the stresses in a foundation, created by the test block, and also from the equality of parameters of slab slenderness both in the model and the field.

The data obtained may be used for the transition from model to field studies.

The following conclusions were drawn from the investigations and from calculation examples

1. The existing method of calculating slabs resting on elastic foundations does not take into account contact friction and, therefore, should be supplemented accordingly. Disregarding friction forces leads to overrated design parameters (moments, deflections, etc) and in some instances (box-type foundations) to entirely wrong results.

2. Due consideration of friction forces may result in a 15 - 35% reduction of calculated maximum tensile stresses in solid slabs, and a 2-5 fold reduction in stresses for box-type structures. In this case the friction forces may also be responsible for changes in sign of the calculated moment.

3. The effect of friction forces increases when lateral additional loads and the operational (water) pressure are taken into account.

4. A solution based on Chebyshev polynomes for finite-rigidity strips on elastic foundations, with friction taken into account, should be considered as the most correct among all the existing approximate solutions. For an extreme case (absolutely rigid strip) the values for reactions of a foundation are close to the exact values determined according to V. M. Abranov's method. By its mathematical approach this method of solution is fairly simple: the stresses in a slab are computed with the help of a four-equation calculation scheme.

The research conclusions may be applied to the design of concrete linings for earth-dam embankments, breakwaters, canals, water-deflecting

aprons, floors of locks and docks, and also for different types of foundations and other structures to be built on an elastic rock base. The application of these calculation methods ensures savings in reinforcement steel for slabs (by 10 to 20% and, in certain cases, even much more).

MISI IMENI V. V. KUIBYSHEV CHAIR OF HYDRO STRUCTURES

Head: Professor M. M. Grishin, Doctor of Technical Sciences, Honored Scientist of the R. S. F. S. R., Fellow of the ASiA of the U. S. S. R.

INVESTIGATION OF STRENGTH AND STABILITY OF COMPOSITE ROCK-FILL DAMS

Scientific Guidance: Professor M. M. Grishin, Doctor of Technical Sciences

Research Team: V. N. Pospelov, Candidate of Technical Sciences, Lecturer
V. I. Shvei, Candidate of Technical Sciences, Lecturer
P. P. Mois, Engineer
N. F. Sal'nikova, Engineer
A. I. Churakov, Engineer

The research has been undertaken as the necessity arose to solve certain problems of design and erection of the Charvak dam (Uzbek S.S.R.) characterized by exceptionally large sizes.

The chief aim of this investigation has been to give consultations to the design institutes SAOGIDEP on problems connected with the design of high-head composite-type dams which have not been as yet designed in the U.S.S.R.

The paper contains a number of final reports, each dealing with an independent problem.

Report No. 1. Literature on the design and construction of more than 20 high-head composite-type dams has been reviewed, systematized and analyzed. The report gives characteristic data of dams (geological conditions, height, location of slopes, volume and distribution of material through the cross section, economical data) which make it possible to draw conclusions on the modern trends in the construction of earth-fill and rock-fill dams.

The report contains illustrations and photographs of some dams and (in the appendix) a summarizing table on 50 dams.

Report No. 2 gives basic data on construction discharge capacity, geological conditions, etc. for different dams, as well as data on saddle flood spillways of the open and tunnel types which are used at 17 high-head rock-and-earth (composite) dams with a height varying from 60 to 226 m, and at 24 high-head plain concrete and reinforced-concrete dams both in the Soviet Union and abroad. The report contains general layouts of hydro developments, drawings of spillways and suggestions for the design of an efficient flood spillway for the Charvak HEP taking into account the specific conditions of the construction site.

Report No. 3 gives basic data on the material and the methods of placing and compacting the impervious elements of rock-fill and earth-fill dams 30 and more meters high, built or designed both in the Soviet Union and abroad. Examples of dams with impervious cores made of natural or

artificial materials are given as well as brief information on mixing technology for artificial filling materials. Size-grading curves for core material, plans of dams, and a summary diagram of the size-grading curves for the material of impervious cores for these dams are attached to the report.

Report No. 4. The report contains information on the pore pressure, conditions of its appearance, and on the ways to reduce it. The report also deals with the principles of design of gages for measuring the water pressure in the soil, as well as drawings for a pore-pressure gage developed by the chair staff and built and tested at the laboratory of MISI imeni V. V. Kuibyshev.

The report has been passed on to the customer for implementation.

INVESTIGATION AND DESIGN OF CONTROL-MEASURING INSTRUMENTS INTENDED FOR ANALYSIS OF STRESSES IN A MASSIVE CONCRETE DAM

Responsible for Research: Professor M. M. Grishin, Doctor of Technical Sciences

Research Team: V. G. Orekhov, Candidate of Technical Sciences, Junior Lecturer
B. I. Komzin, Postgraduate
A. I. Churakov, Engineer, and others

Stresses inside a concrete body are measured indirectly by determining, under field conditions, the deformations in concrete with the aid of transducer gages and by subsequent theoretical computation of stresses. In order to calculate the stresses it is necessary to separate the elastic deformations from the total measured deformations. This is done by eliminating the free, body-type and creep deformations. It has, however, been proved that the elimination of elastic deformation is almost nonachievable and, consequently, the stress pattern determined by this [indirect] method would considerably be distorted. Therefore, the chief objective of this investigation has been to develop an instrument which will render possible the direct measurement of stresses in a concrete body. The development of appurtenant recording devices was also included into the research schedule. The research program included: 1) a review of the foreign and U. S. S. R. experience in the investigation of stresses in concrete; 2) theoretical studies of the working conditions for a foreign element (e. g. a gage) embedded in the concrete; 3) suggestions for the design of gages for measuring stresses in concrete; 4) design details for such a gage; 5) laboratory and field tests of those gages; 6) development of recording devices.

In 1957 transducer models were built and tested in this laboratory. The transducers were tested and calibrated for the measurement of temperatures and loads. The conclusions reached from this research are: 1) the new type of transducer may be used for direct measurements of stresses in concrete; 2) the transducer is almost insensible to changes in temperature provided its dimensions are properly chosen; 3) the transducer is stable in operation, highly sensitive and has linear calibration for loads.

Together with the design of the transducer, highly sensitive electronic recording devices were developed.

In 1958 the transducer was tested under field conditions, and its adaptability for the measurements of stresses in concrete has been fully proved.

MIIVKh IMENI V. R. VIL'YAMS CHAIR OF BUILDING MECHANICS

Head: Professor L. M. Emel'yanov, Doctor of Technical Sciences

EXPERIMENTAL INVESTIGATION OF THE BEHAVIOR OF UNDERGROUND STEEL PIPES

Responsible for Research: K. A. Ksenofontov, Candidate of Technical Sciences

Research Team: V. F. Luppov, Junior Lecturer
I. Yu. Yushmanova, Junior Research Worker

Underground steel pipes for conveying different liquids have found, in recent years a wide application both in the Soviet Union and in foreign countries. This is why the developing of calculation methods for such pipes and the working out of efficient design solutions has become more and more urgent. However, owing to insufficient knowledge of working contions of underground pipes, these aims have not yet been fully achieved. In particular, problems concerning the contact and interaction of the pipe walls with the surrounding soil, stresses in pipe walls, and durability of a pipe are still to be solved.

In the years of 1957-58 laboratory experiments have been carried out with the purpose of studying certain aspects of the behavior of thin-walled, underground steel pipes. The program of this investigation included the determination of the interrelation between the magnitude of a load applied on the surface of the backfill of a thin-walled steel pipe, and the pressure distribution on the pipe surface on the one hand, and deformations and displacement of the pipe wall on the other. At the same time it has been intended to study the effect of wall thickness, fastening (rigidity) rings, height of backfill, and nature of the load on the durability and behavior of the pipe.

The pipe models, 67 cm in diameter and 200 cm long, were made of steel sheets 1.25 mm and 2.00 mm thick. Some specimens were fitted with welded-on bracing rings, the distance between the rings being equal to one or to two [pipe] diameters.

The test stand consisted of a compacted underground pit (chamber) 210×180 cm, 200 cm deep, loading and handling equipment, and auxiliary premises. The longitudinal [retaining] walls of the pit were made of massive brickwork; bulkheads of the prefabricated type were made of steel panels welded from I-beam No. 18 and steel plates 10 mm thick.

The models were laid on a soil layer 46 cm thick (which corresponded to about 3/4 of the pipe diameter). The height of backfill above the curved surface (crown) varied between $\sim 0.5\,D$ (34 cm) and $\sim 1.0\,D$ (68 cm). At the places of contact between the pipe ends and the bulkheads the chamber was fitted with manholes 55 cm in diameter to permit installation of the control measuring equipment inside the models.

The loading equipment consisted of four high-strength channel beams laid on the backfill surface, 12-ton hydraulic jacks and five supports welded from I-beams No. 18 and reinforced with 10 mm thick steel plates. By placing all the four loading beams on the backfill a uniformly distributed load was simulated, while a single loading beam simulated a locally applied load.

The effect of eccentricity of load application on the pipe was checked by applying the axis of the local load at various distances from the pipe axis.

The handling equipment consisted of two overhead trolleys 0.5 t each, traveling on two I-beams No. 22, which were laid along the ceiling and fixed with their ends to the walls of the laboratory building. The trolleys were used for handling the various test-stand elements, weighing up to 350 kg each, and for hauling the sand for the backfill of the trench.

The following parameters related to the measured intensity of load applied on the backfill surface, have been determined:

a) stresses in the pipe wall – by ohmic-resistance strain gages;

b) radial displacement of the model shell – by dial gages calibrated to 0.01 mm;

c) backfill pressure on pipe – by membrane dynamometers;

d) backfill pressure on the bottom and walls of the test chamber – by wire-type strain gages.

The principal conclusions reached from the investigation are:

1. A very flexible steel pipe (which sags under its own weight) acquires a high bearing capacity when buried in the ground. Under a continuous load of 3.33 kg/cm^2 applied to the backfill surface (equivalent to a backfill of about 20 m thickness) the pipe retains its carrying capacity. Losses in the model's stability could be obtained during the test only by applying local stripwise loads.

2. The magnitudes of the critical loads approach those obtained by the E. L. Nikolai formula, developed for pipes laid in elastic media.

3. The pressure acting on pipes is almost uniform.

4. The strength of the shell has only an insignificant effect on the sag of the pipe and on the size of the critical load. When using reinforcing rings, which increase the shell rigidity as much as 200 times, variations in the sagging did not exceed the measurement accuracy (0.01 mm), the critical load not increasing by more than 60%.

In the light of the aforesaid, the following conclusion may be reached:

The application of a large number of strong reinforcing rings on thin-walled underground steel pipes is unnecessary. Two to three light-type reinforcing rings for one pipe are sufficient to protect the pipe during transportation, handling and backfilling. This ensures savings in metal of up to 20 to 30%, simplification of pipe laying, and transfer of highly qualified welders to other jobs.

VNII VODGEO LABORATORY FOR EARTH STRUCTURES AND FIELD INVESTIGATION

Head: I. V. Fedorov, Candidate of Technical Sciences

DEVELOPMENT OF DESIGNS FOR CANAL LININGS

Research by: A. G. Rodshtein, Candidate of Technical Sciences, Senior Research Worker

The work contains a review of U.S.S R. and foreign literature (1946-1958) on existing methods of protecting canal slopes, and on the construction of linings; information concerning the construction of joints between the linings and the sealing agents therefor; description of the construction of impervious linings on the Northern Don'ets-Donbass Canal. A literature survey containing 148 references is attached to the paper.

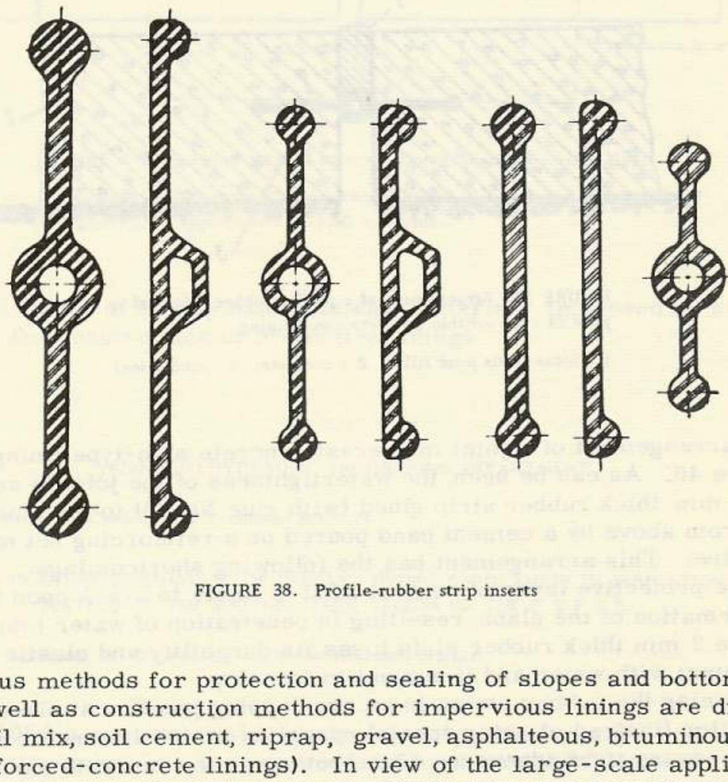


FIGURE 38. Profile-rubber strip inserts

Various methods for protection and sealing of slopes and bottoms of canals as well as construction methods for impervious linings are discussed (e. g. soil mix, soil cement, riprap, gravel, asphalteous, bituminous, plain and reinforced-concrete linings). In view of the large-scale application of precast, concrete and reinforced-concrete slab-type linings, the methods for sealing joints, appurtenant material and application procedures are reviewed.

In foreign practice, profile-rubber strip seals (Figure 38), seals made of P. V. C. (type "Dyuradzhoi") and of other synthetic materials have recently gained wide acceptance. Various compounds made of bitumen and rubber compositions, resins, rosins, oil, etc. are extensively used as joint fillers.

Compounds are divided into various categories according to their application methods: cold compounds placed with a trowel, molten compounds poured into the joint, and compounds fed pneumatically.

In most cases the composition of compounds is protected by the patent law; usually the components are supplied to the construction site by commercial companies.

The study deals with the experience in using rubber seals on linings for the Northern Don'ets-Donbass Canal. The results of these studies revealed the positive and negative aspects of this method.

As can be seen from the design in Figure 39, a profile-rubber strip seal is incorporated into the joint of a monolithic concrete canal lining. The seal, 10mm thick, ensures full watertightness for the whole service life of the rubber. The flexible, central, hollow section of the seal is able to sustain considerable deformations of the concrete lining.

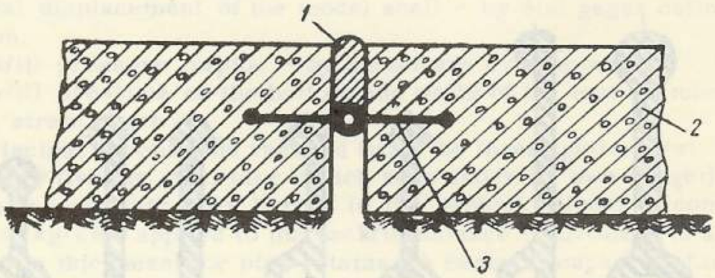


FIGURE 39. Arrangement of a profile-rubber strip seal in a joint of a monolithic concrete canal lining

1 - bituminous joint filler; 2 - concrete; 3 - rubber seal.

The arrangement of a joint in precast concrete slab-type linings is shown in Figure 40. As can be seen, the watertightness of the joint is achieved by a thin 2 mm thick rubber strip glued (with glue No. 88) to the slab and protected from above by a cement band poured on a reinforcing net made of 1 mm wire. This arrangement has the following shortcomings:

a) the protective layer of cement band is liable to crack upon the slightest deformation of the slabs, resulting in penetration of water into the joint;

b) the 2 mm thick rubber strip loses its durability and elastic properties upon contact with water and if subject to freezing;

c) placing the rubber strips in an overlapping position at places of slab intersection (instead of using special crosslike or tee-like rubber sheets) does not ensure tight adherence of the rubber to the concrete surface.

These shortcomings reduce the long-term watertightness of the lining.

The study of relevant literature, available project documentation, and of data on the experience in construction and operation of canals made it possible to establish the following subjects for further research:

1) development of an efficient lining type which could ensure long-lasting operation of canals;

2) increase in the watertightness of linings by further improving the design of concrete-slab joints;

3) development of suitable technology for the manufacture of seals and compounds from new synthetic materials so as to ensure a reliable and long-lasting watertightness of the linings;

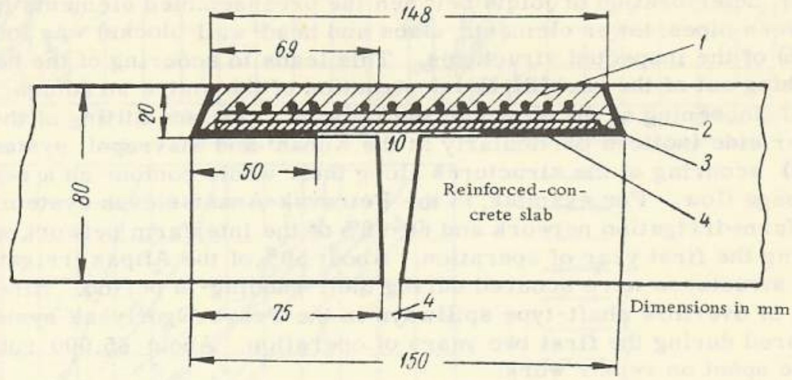


FIGURE 40. Design of joint in concrete linings made of precast slabs

1 - cement-sand mortar; 2 - reinforcing mesh, 3 - rubber sheets; 4 - glue No. 88 applied to a strip 50 mm wide.

4) application of locally available raw materials, improved by artificial means, for the construction of protective linings.

YuZhNIIGIM HYDRAULIC-ENGINEERING DEPARTMENT

Head: I. A. Chuprin, Candidate of Technical Sciences

INVESTIGATION OF BEHAVIOR OF PRECAST HYDRO STRUCTURES IN IRRIGATION SYSTEMS OF THE SOUTHEASTERN REGION OF THE R. S. F. S. R.

Responsible for Research: N. O. Filippov, Senior Research Worker

During 1958 a comprehensive study on design, construction and operation of precast structures has been carried out by the YuZhNIIGIM with the aim to select and further improve the most efficient types of such structures; to improve the quality of construction and of operation of structures; to establish the causes of deformations and to examine the possibility of using the block-type construction method. The following work has been carried out to this end:

a) complex investigation of structures in the irrigation system of the Rostov and Stalingrad Regions and Krasnodar and Stavropol' Territories;

b) hydraulic investigation of assembled tubular structures (pipelines);

c) hydrotechnical investigation of underground structures of different contours.

I. More than three hundred structures of different types, among them 65-70% tubular and 30-35% open-type structures were examined.

The prevailing deficiencies are:

a) scouring of tailwater aprons over a length of 4 to 12 m against a design length of the aprons of 3-6 m;

b) deterioration of joints between the preassembled elements (especially between pipes, tower elements, slabs and head-wall blocks) was found in most (98%) of the inspected structures. This leads to scouring of the head walls, washing out of the backfill, and destruction of the entire structure;

c) loosening of the slopes at the tailwater side and silting of the headwater side (noticed particularly at the Kuban' and Stavropol' systems);

d) scouring of the structures along their whole contour as a result of seepage flow. For example, in the Petrovsk-Anastasievsk system, 30% of the farm-irrigation network and 60-68% of the interfarm network was scoured during the first year of operation. About 50% of the Afipsk irrigation-system structures were scoured during their running-in period. Ninety per cent of overflow shaft-type spillways in the Pravo-Egorlyksk system were scoured during the first two years of operation. About 65,000 rubles were spent on repair work.

The main causes of these failures were found to be the following:

a) inadequate hydraulic regime;

b) insufficient length of underground and lateral structures for seepage drainage and inadequate length of downstream (tailwater) aprons;

c) poor design of joints between elements and especially those in conduits;

d) poor workmanship of the construction work and low consolidation of backfill (bulk weight of backfill in the dry condition, $1.2\text{-}1.4\ t/m^3$; in the natural condition, $1.5\text{-}1.7\ t/m^3$);

e) inadmissible lack of precision in vertical fitting; repeated departure from design specifications; oversimplification.

The conclusions arrived at from the comparison of basic technical and economical characteristics of different types of prefabricated structures are:

1) Introduction of prefabricated structures ensures mechanization of the construction process with concomitant reductions in labor and the amount of work involved by a factor of two and two to six respectively.

2) Tubular-type pressure spillways developed in 1956 at the Yuzhgiprovodkhoz are the most efficient structures for stream-flow junction between the headwater and tailwater sides. The construction cost is 1/2 to 1/3 of the amount of construction work, and 1/3 to 1/5 of that of conventional structures; labor expenditure is smaller by a factor of 3.5 to 6.2 than that required for a monolithic construction. This type of structure most satisfactorily suits the general scheme of surrounding structures and it is universal and reliable in operation (Figure 41).

3. The most rational amongst the level-regulating locks are those of the open-type ("RO" designed by Ukrgiprovodkhoz; "ROV" by the Pyatigorsk branch of the Yuzhgiprovodkhoz, and others).

The "RO" locks are simple in design, made of large blocks and reliable in operation.

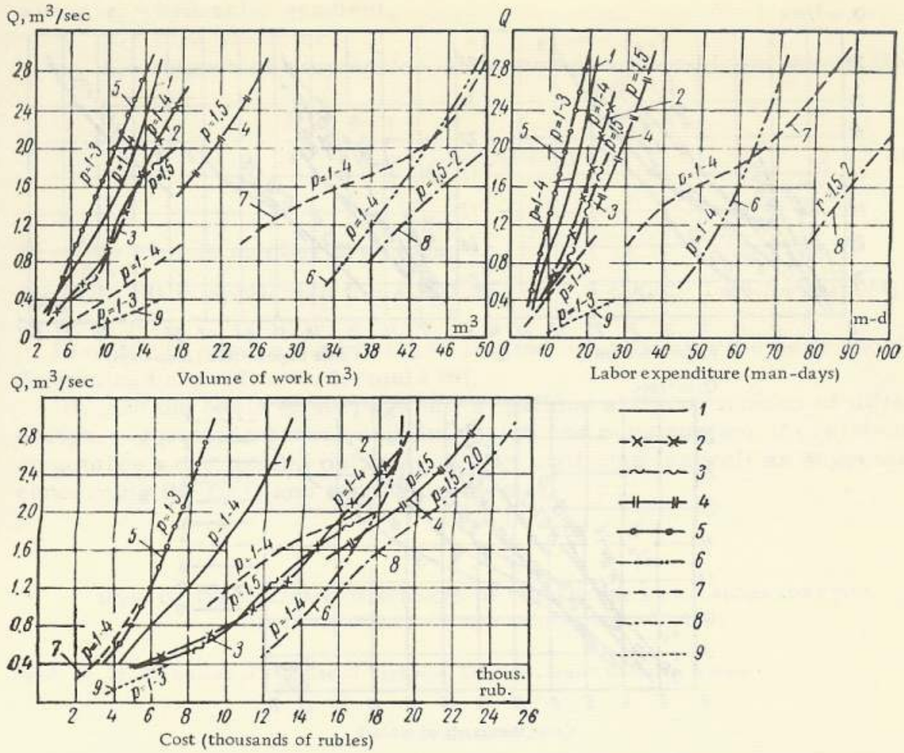


FIGURE 41. Technical and economical data for spillways and chutes of the prefabricated type:

1 - tubular pressure spillways (1952); 2 - tubular-semipressure spillways (1952); 3 - tower-type spillways (1953); 4 - open chutes (1954); 5 - tubular pressure chutes (1956) of the monolithic type; 6 - open chutes (1931); 7 - idem (1939); 8 - idem (1954); 9 - cantilever-type chutes (1939); P = overflow height (1-4 m).

A detailed analysis of operation of various types of structures is given in the paper.

II. Hydraulic investigations have been carried out on seven tubular water outlets (designed by the Gidroproekt) on which K. S. Glubshev's current meters have been installed.

The study has been carried out to determine the relationship between the extent of gate opening and the coefficient of overflow-shaft resistance $\zeta_{sh}=\varphi\left(\frac{h_g}{d_t}\right)$. As a result, an empiric relation has been obtained

$$\zeta_{sh}=\frac{1.8}{\left(\frac{h_g}{d_t}\right)^{2.7}}. \quad (1)$$

The working distance between the VDG-56 current meter and the gate is found by the formula

$$C_p \approx 12\, d_t - 0.5\, h_g. \quad (2)$$

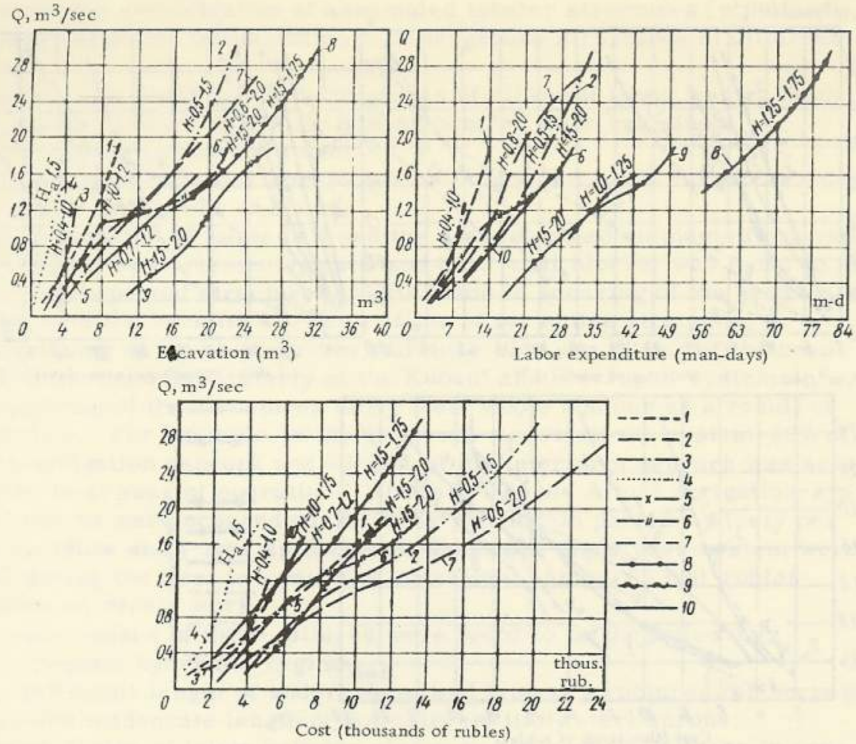


FIGURE 42. Technical and economical characteristics of regulating locks:

Prefabricated locks: 1 - open "ROV" (1953); 2 - open "RO-11-A" (1954); 3 - open (1953); 4 - open "RO" (1953); 5 - tubular "RT" (1952); 6 - idem – with monolithic head walls; 7 - tubular "RTV" (1954); monolithic locks: 8 - open "RO-11-M" (1954); 9 - open (1931); 10 - tubular (1931); H = pressure on the sill; Q = discharge for L = 0.1m.

The discharge coefficient of prefabricated tubular-type pressure spillways (designed by the Yuzhgiprovodkhoz 1958) may be found from empirical formulas derived on the basis of an investigation carried out on three different structures:

a) for the pressure discharge regime

$$\mu = 2.27\sqrt{(K_0 - 1.35)\frac{d_t}{z_0}}; \tag{3}$$

b) for the semipressure discharge regime

$$\mu = 2.90\,(K_0 - 1)\sqrt{\frac{d_t}{z_0}}, \tag{4}$$

where z_0 = hydraulic gradient;

d_t = pipe diameter;

K_0 = degree of submersion of the entrance found from the relation

$$K_0 = \frac{H}{d_t} = 1.35 + 0.1 \text{ Fr}. \tag{5}$$

$\text{Fr} = \frac{\alpha v_t^2}{g d_t}$ Froude number for pipes.

Formula (3) is applicable for $K_0 > 1.35$; at $1.0 < K_0 < 1.35$ formula (4) should be used.

In order to ensure the pressure regime of spillways, the head should be determined according to formula (5).

III. On the basis of seepage investigations at 25 structures of different design, suggestions concerning the design and construction of prefabricated structures were passed on to the design institutes (as well as suggestions concerning the first and second problems).

IEVKh OF THE ACADEMY OF SCIENCES OF THE KIRGHIZ S.S.R. LABORATORY FOR THE DEVELOPMENT OF NEW IRRIGATION METHODS

Head: M. M. Kabanov, Candidate of Technical Sciences, Senior Research Worker

PAVED CANALS

Responsible for Research: M. I. Nazarov, Junior Research Worker

Scientific Advisers: N. A. Yanishevskii, Doctor of Technical Sciences, Associate Member of the Uzbek Academy of Sciences
M. M. Kabanov, Candidate of Technical Sciences, Senior Research Worker
A. V. Troitskii, Candidate of Agricultural Sciences, Lecturer

Steep-slope (0.01) canals in hilly and mountain regions of Kirghizia are exposed to an extensive erosion by fast-flowing water. Usually erosion penetrates the underlying strata until it reaches the highly pervious gravel bed.

The operation of a canal under such conditions leads to washing away of its banks and to meandering of its channel. Such a canal looks more like a flood plain (Figure 43), and the resulting seepage losses attain critical values. Under these conditions, at a discharge of 1 m^3/sec, seepage losses attain 9,6%, thus reducing the efficiency of irrigation systems in hilly regions (0.5-0.4 and less).

In Kirghizia, methods for protecting canals from scouring and seepage losses, by paving them with boulders, have recently gained wide acceptance. At present more than 500 km of paved canals are in operation (Figure 44), however, a detailed hydraulic investigation has not yet been carried out. Therefore, owing to certain inaccuracies in hydraulic computations, most of the paved canals operate below their rated discharge capacity.

FIGURE 43. Sel'skii Canal of the Alalidin River irrigation system before its redesign (slope i = 0.025)

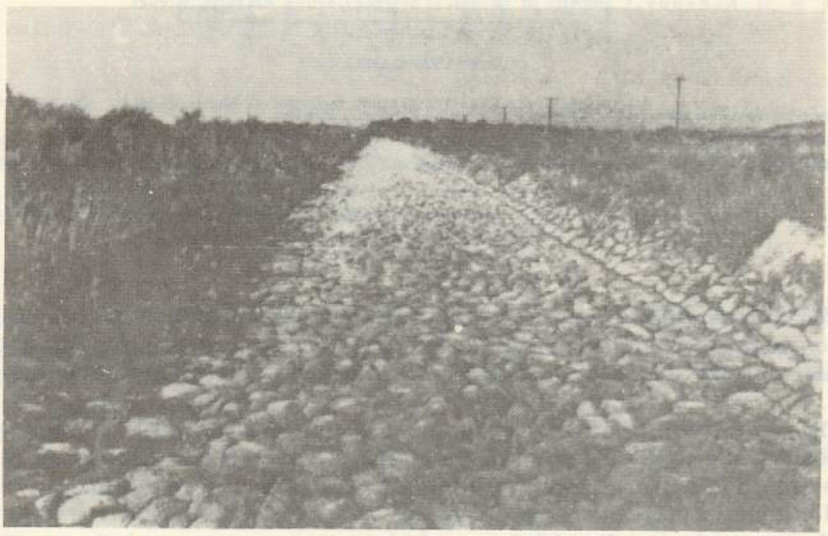


FIGURE 44. Paving of canals with boulders

Between 1954 and 1958 hydraulic researches on paved canals have been conducted by the staff of the Institute with a view to developing and presenting suggestions for the design of such structures.

These investigations were intended to determine the following parameters:

1) coefficient of roughness and velocity coefficient C for paved canals;
2) velocity distribution in the cross section of paved canals;
3) admissible and erosive velocities in paved canals.

The coefficient of roughness, the velocity coefficient C, and the velocity profiles were investigated in fourteen sections of operating canals of different parameters (b, h, i, α, Δ_{av}). The test sections remained under observation during the whole season of operation. The flow velocities required for discharge calculations were measured by Zh-3 type current meters. The velocity diagrams were plotted from the results of velocity measurements (at 10 to 15 points along the vertical) by means of the Rehbock tube. The slope of the water surface was determined by measuring the water level in stilling (surge) tanks located at the boundaries of the test section. Similar investigations were also carried out on two experimental canals paved with layers of different thickness (0.05-0.30 m). During the whole investigation period more than 600 readings were taken in operating and in the experimental canals and more than 300 velocity-distribution diagrams were plotted.

After processing of the test results, a function for the coefficient C was obtained:

$$\frac{C}{\sqrt{2g}} = 2.8 + 7.5 \log\frac{R}{\Delta_{av}}, \qquad (1)$$

where C = velocity coefficient;
R = hydraulic radius
Δ_{av} = average height of stone projections of the first paving.

Tables were plotted showing the relationship between the roughness coefficient and the methods of placing the stones in the lining. The results obtained differ to a considerable extent from the existing practice.

By processing data on velocity distribution in the vertical section, a simple relationship was obtained.

$$U = 1.33\, v\eta^{1/3}, \qquad (2)$$

where U = velocity at a given point at a distance y above the bottom;
$\eta = \frac{y}{H}$ = relative depth;
v = mean velocity along the vertical.

Erosive velocities have been determined on two special experimental canals successively paved by stones increasing in diameter from 0.05 up to 0.30 m. The discharge in the first experimental canal was 2.5 m^3/sec, and in the second, 3.0 m^3/sec. By correspondingly increasing the discharge, the lining was brought three times to a state of impending failure. In addition, failures in the paving of two operating canals were also recorded. Thirty experiments were carried out on special test canals for different b, d, i, Δ_{av}, m.

By processing the test results, a relationship for the erosive velocity was obtained

$$v_{er} = 0.0507\zeta C^{1/3}\sqrt{d(\sigma-1)}, \qquad (3)$$

or in a more developed form

$$v_{er} = 0.0507 \frac{Q^{0.12} (\beta + 2m')^{0.09} \sqrt{d(\sigma - 1)}}{n^{1.21}\ i^{0.0606} (\beta + m)^{0.21}}, \qquad (4)$$

where v_{er} = erosive velocity (mean value) for cross section;

$\xi = \sqrt[3]{\frac{\beta + 2m'}{\beta + m}}$ form factor of the river channel;

Q = discharge;

n = roughness coefficient;

i = slope;

$\beta = \frac{b}{n}$;

m = inclination angle of slopes.

To facilitate calculation of v_{er} a special chart (Figure 45) has been plotted.

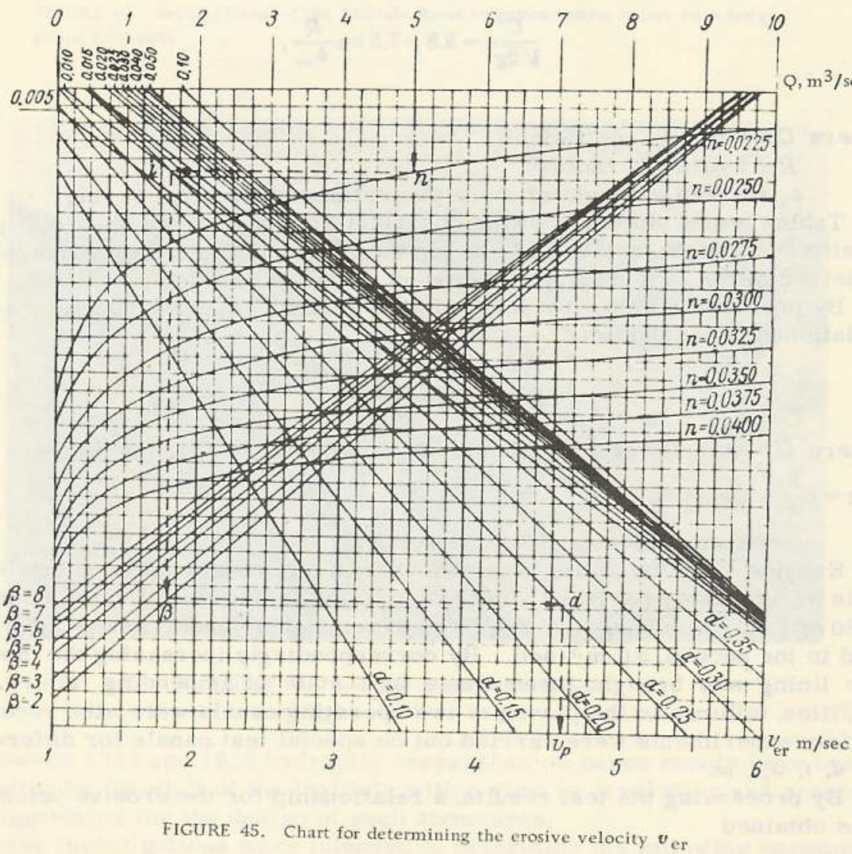


FIGURE 45. Chart for determining the erosive velocity v_{er}

By introducing into the formula (4) safety factors K_0 and K_s which depend on the type of canal used (main canal, distribution canal, etc.) as well as the discharge capacity and the quality of subgrade of the paving, a relationship is obtained for the permissible flow velocities in paved canals

$$v_{per} = K_0 K_s v_{er} .$$

Paved canals designed on the basis of these formulas are reliable, and their construction cost may be reduced by 25 to 30% compared with that of previously built canals. In addition, maintenance and repair costs are reduced by 80 to 90%. The results of these investigations are being used in the design, construction and operation of paved canals in hilly and mountain regions of Kirgizia. They may also be successfully applied in other similar regions, where the prevailing conditions and the availability of suitable paving materials make it possible to apply this type of revetment to eliminate scouring and losses of water in irrigation canals.

The paper was published in 1958 by the Academy of Sciences of the Kirgiz S. S. R.

KIIVKh CHAIR FOR HYDRAULIC STRUCTURES

Head: Professor V. V. Aristovskii, Doctor of Technical Sciences

ECONOMIC CONSIDERATIONS IN THE CONSTRUCTION OF SANDY-SOIL-FILLED DAMS WITH NONPROTECTED UPSTREAM SLOPES

Responsible for Research: E. S. Tsaits, Junior Lecturer

Research Team: E. S. Tsaits, Junior Lecturer
A. D. Petrash, Engineer

The study presents a method for the calculation and design of earth-fill dams built of noncohesive sandy soils and having nonprotected upstream slopes.

According to the suggested method, the characteristic parameters for a nonprotected slope resistant to water-wave action and computed from empiric functions, have higher absolute values than those derived earlier by other investigators. Therefore, the chief objective of this work has been to work out a characteristic parameter for the evaluation of the expediency of replacing the usual type of dam-face protection by gently sloping nonprotected faces, calculated on the basis of the method suggested in this study.

The evaluation of economic effectiveness of earth-fill dams with nonprotected upstream faces is based on the parameter "E" which represents the ratio of construction cost of an earth-fill dam with nonprotected upstream faces to that of a dam with slope facings. The types of facings being studied are dry tree branches, stone and concrete revetments.

The investigations have been conducted under conditions of the two-dimensional problem, i. e. the volume of facing work per running meter of structure was taken into account as well as variations in the formation of

wind-driven waves on the storage reservoir and fluctuations in the construction cost of different kinds of works in dependence on the method of their performance, the hauling distance of building materials, location of the construction site, etc.

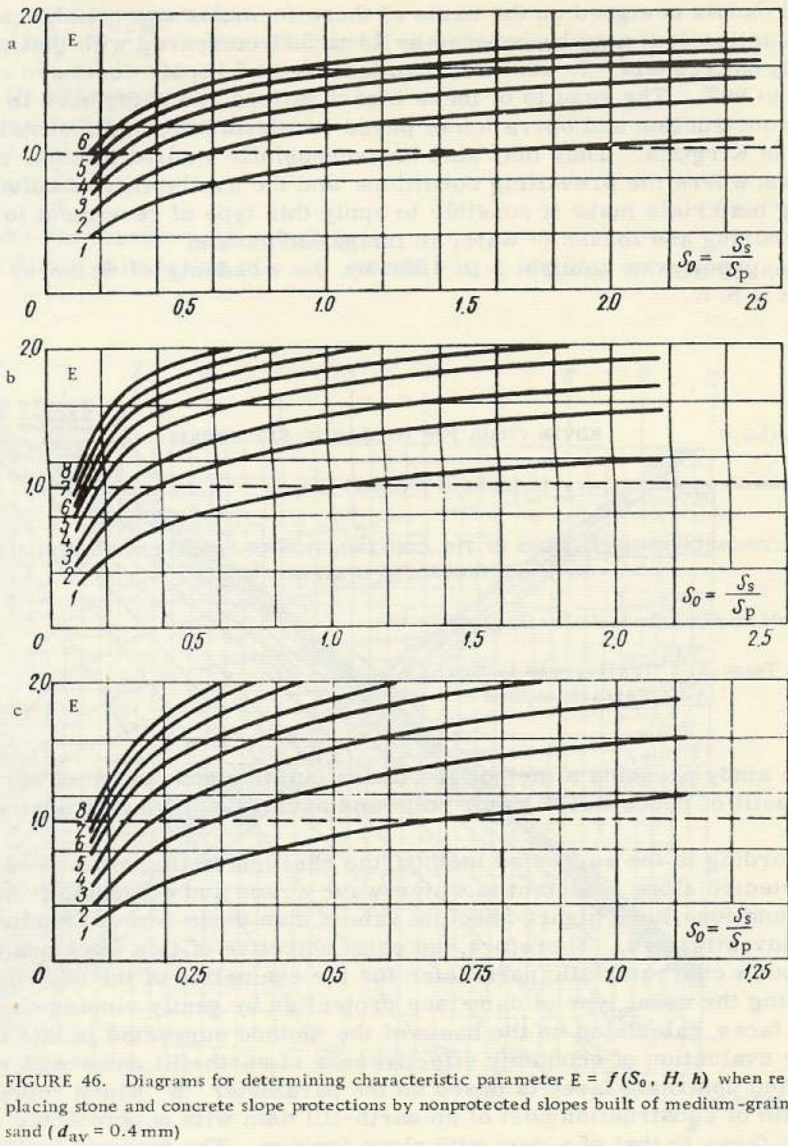


FIGURE 46. Diagrams for determining characteristic parameter $E = f(S_0, H, h)$ when replacing stone and concrete slope protections by nonprotected slopes built of medium-grained sand ($d_{av} = 0.4$ mm)

a – height of wave $h = 0.1$ m; b – $h = 0.2$ m; c – $h = 0.3$ m; d – $h = 0.5$ m; e – $h = 1.0$ m; f – $h = 1.5$ m; g – $h = 2.0$ m; h – $h = 2.5$ m; 1, 2, 3. . . . curves corresponding to height of structures $H = 1.0$; 2.0; 3.0 m; etc.

The cost of different works has been estimated on the basis of unit prices in force in the Ukr. S. S. R. with allowance for local conditions.

The dimensions of the protective slope linings were taken as the arithmetic mean of results obtained from the formulas derived by Gidroenergoproekt and the IGiG of the Academy of Sciences of the Ukr. S. S. R.

The results are given in the form of diagrams (Figure 46) for the mean grain diameter of noncohesive soil d_{av} = 0.4 mm. For grain diameters different from the above values, the following conversion coefficients K should be applied:

Mean grain diameter of noncohesive soil d_{av}, mm	0.2	0.4	0.8	1.6
K	1.09	1.0	0.92	0.86

These graphs make it possible to determine to a first approximation the characteristic parameter E, provided the following rates are given: ratio $S_o = \frac{S_s}{S_p}$ of the cost of one cubic meter of soil to that of one square meter of protective lining of calculated dimension, as well as the wave height h, in m, mean grain diameter of sandy soil d_{av}, in mm, and dam height H, in m.

The paper includes suggestions for the determination of the parameter E under conditions of the three-dimensional problem. Capital investment and annual maintenance costs for both types of structures have not been compared. These expenditures will be determined later on, on the basis of observations which are now being carried out on dams with nonprotected upstream slopes.

From these investigations it may be concluded that it would be expedient to replace slopes protected with dry tree branches by nonprotected slopes only if the height of the structure does not exceed 2 m.

The replacement of stone and concrete slope protections by nonprotected sandy slopes would be justified for certain cases and for a height H above 10 m, provided, however, that the cost of earthwork be low enough. In structures lower than 10 m, the savings amount to 20 to 30% of the cost of protected slopes.

VNIIGS LABORATORY FOR HYDRAULIC STRUCTURES

Acting Head of Laboratory: S. P. Shik, Engineer

REINFORCED-CONCRETE ROUND, HOLLOW PILES

Responsible for Research: A. I. Prudentov, Engineer

Research Team: Professor V. I. Petrashen', Doctor of Technical Sciences
S. P. Shik, Engineer
A. Ya. Serebro, Engineer
A. A. Sokolov, Engineer

The objective of this investigation has been to introduce into hydrotechnical, industrial and housing construction, precast reinforced-concrete hollow piles provided with the usual type of nonprestressed reinforcements, or prestressed with high-strength wire. The piles are driven into the ground with their open bottom end, so that the soil, accumulated in the interior, forms a core which is left inside the pile.

The main problems of the investigation were:

1) examination of the existing types of piles and design of new types of hollow piles;

2) study of methods and optimum technology for manufacturing and driving hollow piles;

3) study of the pile-driving processes and of the behavior of hollow earth-core piles, development of tentative methods for the determination of load-carrying capacity of vertically loaded piles;

4) development of methods for calculating the resistance to cracking of a round cross section subjected to bending stresses, as well as of sufficiently simple and accurate formulas for determining the bending moments corresponding to the appearance of the first crack. This study is intended both for piles with nonprestressed and with prestressed reinforcement, with subsequent checking of the formulas obtained;

5) comparison between the estimated over-all cost of hollow piles and the standard square-cross-section solid piles;

6) recommendations for the use of precast piles in construction projects.

The study included the following experimental work: casting the hollow piles of an outside diameter varying from 0.5 to 1.2 m and a length from 6 to 24 m, and driving the piles into different types of soils; bending test on special benches of single and coupled piles of an outside diameter varying from 0.5 to 1.2 m; measurement of deformations with electric strain gages; extensometric determination of stresses developing in the walls of hollow piles driven into the soil by vibration; static loading tests; laboratory testing of different pile cutting-knife models; testing horizontal loads on hollow-pile models intended for use in mooring structures.

In addition, a considerable number of various casting molds (formwork) and fixtures have been designed and fabricated by the VNIIGS; among them, a compressible driving mandrel designed by V. I. Petrashen', and a driving cap developed by A. I. Prudentov for static-load tests and the driving of hollow piles by pressing should be especially mentioned.

The Central Design Office of the Administration for mechanization of *special and erection Works of the Minstroi RSFSR designed for the Institute* a shuttle-type machine for prestressing the reinforcing wire.

As a result of these investigations the following problems were solved:

1) design of hollow piles reinforced with nonprestressed steel bars;
2) design of prestressed hollow piles using high-strength wire (Figure 47);

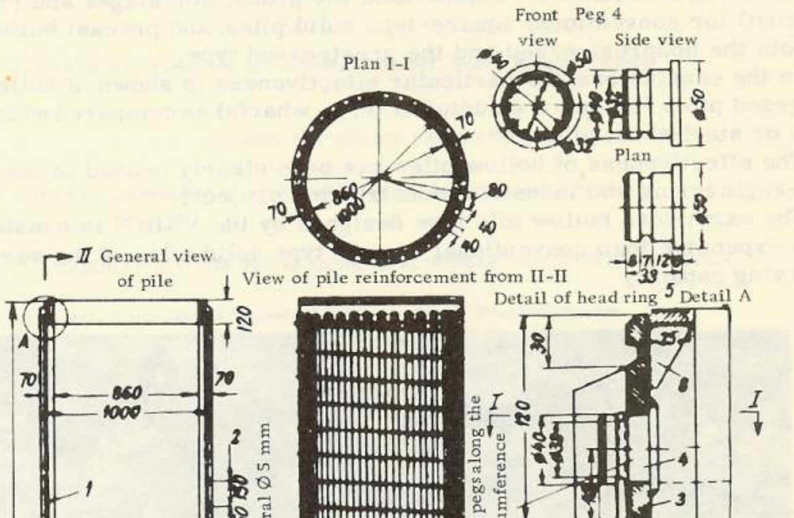


FIGURE 47. Hollow pile prestressed with high-strength wire

1 - longitudinal high-strength 5 mm gage wire; 2 - spiral coil diameter 5 mm; 3 - head ring; 4 - pegs laid around the circumference; 5 - horizontal ring; 6 - diaphragm; 7 - ring-shaped cutter; 8 - cone.

3) technology of field and plant manufacturing of hollow piles with over-all mechanization of the processes involved, the most labor-consuming operations, like that of prestressing the piles, being done on the shuttle-type machine (Figure 48);

4) approximate method for determining the vertical-load carrying capacity of hollow piles with an earth core in clayey and sandy soils;

5) calculation method for the crack resistance of hollow piles under bending stresses for both the nonprestressed and prestressed types of piles;

6) brief suggestions for manufacturing and driving the hollow piles.

The work includes cost comparison (by production stages and raw-material cost) for conventional square-type solid piles, and precast hollow piles of both the nonprestressed and the prestressed type.

In the conclusions, the particular effectiveness is shown of hollow prestressed piles for hydro structures (e. g. wharfs) as compared with square-type or steel-sheet piles.

The effectiveness of hollow piles has been clearly proved in various hydro-engineering and industrial construction projects.

The earth-core hollow pile type designed by the VNIIGS is considerably less expensive than conventional square-type solid piles of the same load-carrying capacity

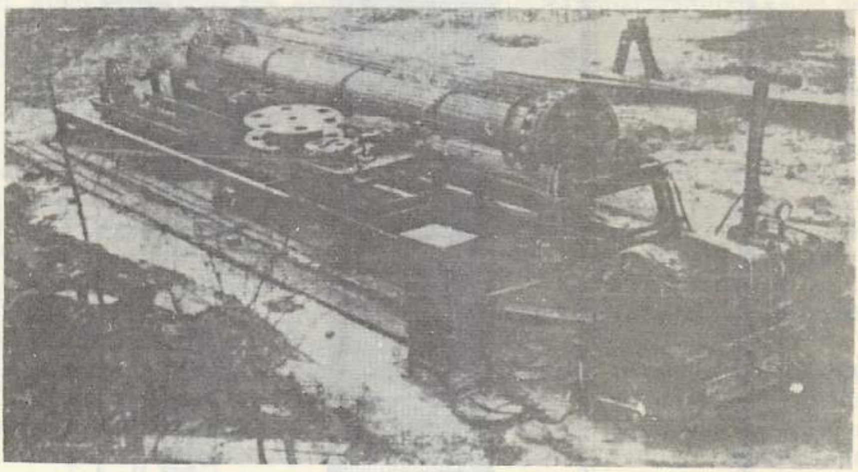


FIGURE 48. Shuttle-type machine for prestressing the reinforcement of hollow piles

For hollow piles reinforced by steel bars without prestressing, savings in steel amount to 25-70% depending on length and diameter of the pile; for prestressed piles, reinforced with high-strength wire, the savings amounted to 50% compared with steel expenditure for nonprestressed solid square-type piles.

The design and calculation methods developed by VNIIGS for hollow piles have been favorably received by numerous design institutes.

Hollow piles of the VNIIGS design are actually being used in some construction projects. Some of the structures built on hollow piles are already in operation and others are still under construction.

PART II

STABILITY OF HYDRO STRUCTURES AND THEIR FOUNDATIONS

VNIIG IMENI B. E. VEDENEEV DEPARTMENT OF SOILS AND FOUNDATIONS OF HYDRO STRUCTURES

Head: P. D. Evdokimov, Candidate of Technical Sciences, Senior Research Worker

SOIL MECHANICS LABORATORY

Head: L. D. Aptekar', Candidate of Technical Sciences, Senior Research Worker

LABORATORY FOR PHYSICOCHEMICAL RESEARCH AND SOIL ANALYSIS

Head: B. F. Rel'tov, Candidate of Technical Sciences, Senior Research Worker

EXPERIMENTAL INVESTIGATION OF THE INTERNAL FRICTION OF SOILS AND THEIR SHEAR RESISTANCE

Research Team: P. D. Evdokimov, Candidate of Technical Sciences, Senior Research Worker
B. F. Rel'tov, Candidate of Technical Sciences, Senior Research Worker

An unsatisfactory aspect of our present knowledge of this subject is that the values obtained for the linear parameters C and φ of the Coulomb equation vary according to the soil-testing method used to determine them. For this reason the VNIIG soil-mechanics laboratory is working out a new theory based on the Boussinesq equation, on the state of stress and the deformation of sandy soils.

For cohesive saturated clay soils it is important to establish the influence of pore-water pressure on the shear resistance of the soil.

The results of the investigations to date are as follows:

1. Differential equations in metric coordinates for the state of stress and soil deformation were derived from the Boussinesq hypothesis (the corresponding equations in rectangular coordinates have been published previously).

2. A solution was obtained to the problem of the state of stress and the deformation of a thick-walled cylinder of soil subjected to normal and shear loads applied to the inner cylinder surface. The solution was based on soil tests carried out in a new testing apparatus (Figure 49).

3. Trials carried out with the new apparatus showed that improvement of a number of construction details was required.

4. The experimental investigation was completed of the shear resistance of sandy subsoils of hydro structures, using the method of rigid dies with the small soil-laboratory flume, at different depths of the die base.

These experiments showed that the shear resistance of the soil to the die penetration can be expressed by the linear Coulomb equation within the limits

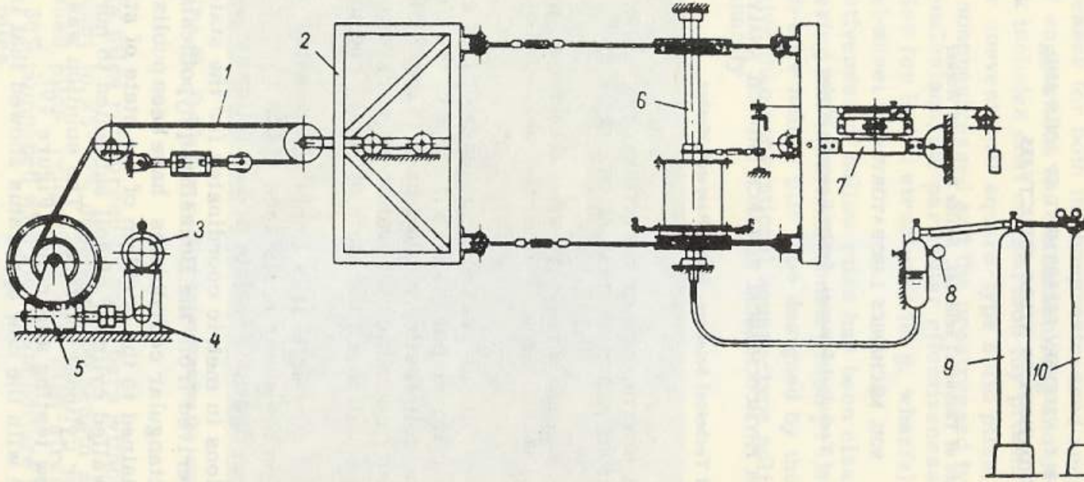


FIGURE 49. Apparatus for studying friction in soils

1–traction cable; 2–traction frame; 3–electric motor; 4– reduction gear; 5–winch; 6–shaft; 7–dynamometer; 8–manometer; 9–buffer cylinder; 10–compressed-air cylinder.

of the critical modeling number $N_{\sigma_{\rm cr}}$), but that the values of the linear parameters are dependent on the depth of the die base.

5. The values of the parameters φ and C having been determined by various methods and under different limit conditions, a hypothesis is offered to explain the variation in the values of these parameters for different deformation conditions of sandy soils. The report includes illustrations of the apparatus used in the different experiments.

6. The experimental investigation was completed of the shear resistance of hydrostatically suspended clay subsoils of hydro structures, using the method of rigid dies with the large soil-laboratory flume, and measuring the pore-water pressure in the clay soil.

This investigation disclosed the mechanism of the shear resistance of clay subsoils, both in a stabilized and in a nonstabilized state. The shear resistance of nonstabilized clay soils is dependent, not on the normal load, but only on the soil density. The shear resistance of stabilized clay soils follows the Coulomb linear equation, the values of the parameters C and φ being determined by the shear test of the soil sample.

7. A number of experiments were carried out to determine the values, for clay soils, of the parameters C and φ. The direct shear and triaxial compression tests were used, under conditions of the so-called "closed" and "open systems". The experiments gave different C and φ values for the soil under different test conditions, and they revealed the influence of the pore-water pressure. These experiments are continuing.

The laboratory for physicochemical research and soil analysis also carried out investigations on the nature of the shear resistance of soils, based on modern concepts of the molecular interaction of soil particles and pore-water, and of the structure-formation processes involved.

In the first stage of the investigation, methods were worked out for the experimental determination of various mechanostructural characteristics of soils under the influence of external normal loads.

Two laboratory techniques were worked out, one based on the principle of the aggregation of a soil prism, and the other on the rotary oscillation of a vertical rod (oscillator) immersed in the soil sample. In addition, methods were worked out for: 1) the determination of the shear-resistance characteristics $\tan \varphi$ and C of soft soils to a shear load varying according to a harmonic law, and 2) the determination of the safety factor of soils subjected to shear, this method being based on the measurements of the shear modulus.

The preliminary investigation of the mechanostructural properties of noncohesive soils established the following:

1. When the static shear stress exceeds the yield strength (coincident with the arbitrary elastic limit) but is still below the ultimate shear stress, the rate of deformation gradually decreases in noncohesive, as opposed to cohesive, soils.

2. The initial shear modulus, which characterizes the disturbance of the bond between the soil particles at the moment when the shear load is first applied, has a low value. With continued application of the load, the shear modulus increases. It follows that, in regard to the shear strength of noncohesive soils, the beginning of shear load application is the most dangerous moment.

3. The disjoining forces acting at the contact areas of the particles of noncohesive soils have no appreciable effect on the strength of the bond

between the particles, which remains constant from the moment contact is established.

4. It is suggested that the decrease in the rate of deformation of noncohesive soils subjected to a constant shear load, and the parallel increase of the initial shear modulus, are related to the gradual increase of the number of contacts between the soil particles resulting from the arrangement of the particles within the mass becoming changed as the shear deformation develops.

OPERATION OF THE MINGECHAUR HYDRAULIC-FILL DAM (ANALYSIS OF OBSERVATIONAL DATA, AND CONCLUSIONS)

Research Team: T. F. Lipovetskaya, Group Engineer
Yu. S. Bol'shakova, Candidate of Chemical Sciences, Junior Research Worker

The present work was begun in April 1957. During 1958 instrument readings were taken of the pore-water pressure. The observations showed that the consolidation of the soil in the core was almost complete. The instruments were functioning satisfactorily and there were no mechanical failures in 1958.

To study the piping phenomena taking place in an earth dam, quantitative analyses were made of samples of the drainage water which showed that the amount and the composition of the salts in this water was substantially the same as in 1957. Only in isolated cases were there small decreases in the quantity of easily soluble compounds such as chlorides. The maximum amount by which the content of salts of the drainage water exceeded that of the reservoir water remained at about 1.0-1.2 g/liter.

In addition, observational data on the seepage conditions and the dam settlement were analyzed and summarized.

The hydraulics department of the Mingechaur HEP carried out weekly measurements of piezometric levels and of the seepage flow through the dam. Control measurements were made in cooperation with a representative of the Institute. The data obtained are indicative of stable seepage conditions in the foundation soil of the dam.

VNIIG IMENI B.E. VEDENEEV SOIL-MECHANICS LABORATORY

Head: L.D. Aptekar', Candidate of Technical Sciences, Senior Research Worker

INVESTIGATION OF THE STRENGTH AND DEFORMABILITY OF DAM FOUNDATIONS AND THE STABILITY OF GRAVITY DAMS BUILT ON ROCK AND SEMIROCK* FOUNDATIONS

Responsible for Research: P. D. Evdokimov, Candidate of Technical Sciences, Senior Research Worker

The following formulas are at present used to compute the stability of gravity dams on rock foundations:

* [Semirocks are rigid soils of a very high degree of lithification.]

A) in the Soviet Union:

$$\eta = \frac{\Sigma N \tan\varphi}{\Sigma E} \tag{1}$$

B) in the U.S.A.:

1) Bureau of Reclamation:

$$\eta = \frac{cF + \Sigma N \tan\varphi}{\Sigma E} \tag{2}$$

2) Engineering Corps:

$$\eta = \frac{rcF + \Sigma N \tan\varphi}{\Sigma E} \tag{3}$$

The symbols in formulas (1),(2),(3) have the following meanings:

η = margin factor of the resistance of the structure against shear;

F = area of the base of the structure, in m^2;

ΣN = sum of normal forces acting on the base of the structure, in tons;

ΣE = sum of shear forces acting on the base of the structures, in tons;

$\tan\varphi$ = coefficient of friction between the concrete and the rock foundation;

c = adhesion, i.e. resistance against shearing in the plane of contact between the structure and the foundation, in t/m^2;

r = ratio of the average to the maximum shear stress, taken as 0.50.

Using the above formulas to compute the external loads for a normal section, the following values are obtained for the safety factor of shear resistance:

a) formula (1) $\eta = 1.00 - 1.05$;

b) formulas (2) and (3) $\eta = 4 - 5$.

The literature does not give standard directives for determining the values of φ and C. The fact that both low (1.0) and high (4 to 5) values of the safety factor are in use shows that our knowledge of this important problem is incomplete.

The work on the present project proceeded mainly on experimental lines, with theoretical treatment of the model analysis of states of stress and deformations in structures and their foundations.

During the preceding years the laboratory designed and built a number of apparatus and testing machines:

1) apparatus designed by I. G. Goncharov for the determination of the strength of artificial and natural stone materials for different states of stress (Fig. 50);

2) testing machine designed by I. G. Goncharov for studying the resistance to shear and sliding of stone samples of parallelepiped shape, along non-fractured (shear) and fractured (sliding) planes of contact;

3) apparatus designed by B. N. Barshevskii and R. A. Shiryaev for determining the shear resistance of small ($d = 7 - 12$ cm) rock drill cores along differently orientated natural cracks (Figure 51);

4) large testing machine for studying the shear resistance of rocky soils by the method of equivalent materials; the shear resistance along

cracks of large (d = 0.90 – 1.00 m) calyx-drilled rock cores; and the shear resistance along the concrete – rock contact plane (Figure 52);

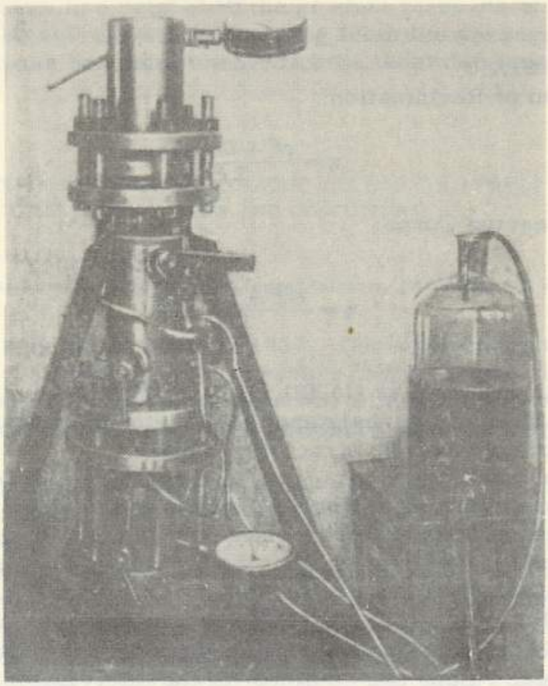


FIGURE 50. Apparatus for determining the strength of natural and artificial stone materials in different states of stress

5) field testing machine for studying the resistance against sliding of concrete structures on a rock foundation, making use of concrete blocks concreted to the rock in underground mine workings, such as mining galleries;

6) field testing machine for studying the resistance to sliding of concrete structures on a rock foundation, using blocks concreted to the rock in the open foundation excavations of the structure;

7) field testing machine for studying the deformability of naturally fractured rock subsoil, by means of hydrostatic pressure on the walls of calyx-drilled boreholes of 90 – 100 cm diameter (Figure 53).

The most important results of the experimental investigation of the strength of rocky soils, and the resistance and deformability of rocky soils and fractured rocky subfoundations of hydro structures, may be summarized as follows:

1. The strength of stone materials depends on the nature of the stress. Under conditions of triaxial compression, the strength of stone materials, at normal stresses of between 30 and 50 kg/cm^2 can be satisfactorily represented by the straight-line envelope of the Mohr circles. Under stress

conditions involving also tension, the relation expressing the strength of stone materials is more complicated, and the shear resistance is sharply reduced.

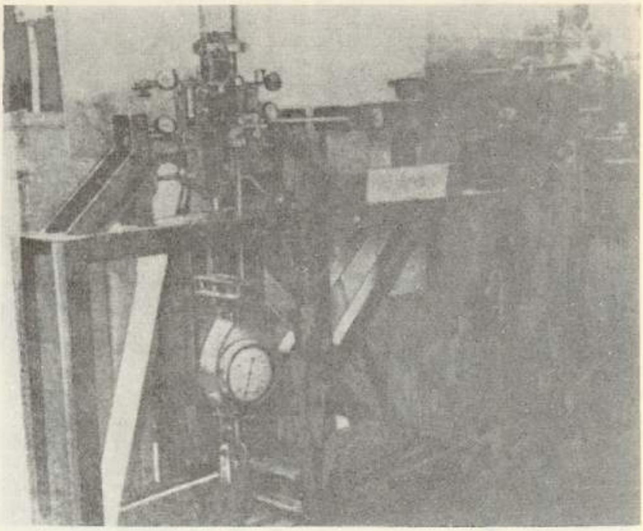


FIGURE 51. Apparatus for determining the shear resistance along rock cracks

2. It is essential to distinguish between, first, the resistance to shear rupture at the contact plane between the concrete and the rock, and, second, the resistance to sliding following the disruption of the contact. Resistance to shear and resistance to sliding can both be expressed by the Coulomb linear equation, but the values of the parameters C and φ in the one case will be different than those in the other case.

3. The shear resistance along the concrete – rock contact plane in the experiments of I. G. Goncharov depends on the roughness of the original rock surface, and it is considerably higher than the resistance to shear or to sliding of concrete blocks on a solid-rock foundation. Introducing in formulas (2) and (3) the shear-resistance values for the concrete – rock contact plane, obtained from tests in the Goncharov apparatus, we find that, for certain roughness conditions of the contact plane, the safety factor taken in the case of gravity dams constructed in the U. S. A. may reach the value $\eta = 4 - 5$. If we take the values of C and φ, determined from shear tests of concrete blocks on a rock foundation, the safety factor φ will be less than $4 - 5$.

4. The difference between values of C and φ for the shear of concrete – stone contact planes is obtained by laboratory tests on concrete-stone samples, and field tests made on concrete blocks built on an actual rock subsoil. This may be explained by the state of the latter, and by the different nature of the stress in the two cases. It must be assumed that in the second

case the contact is broken as a result of the block being torn off the foundation, i.e. that the state of stress includes tension (see 1. above).

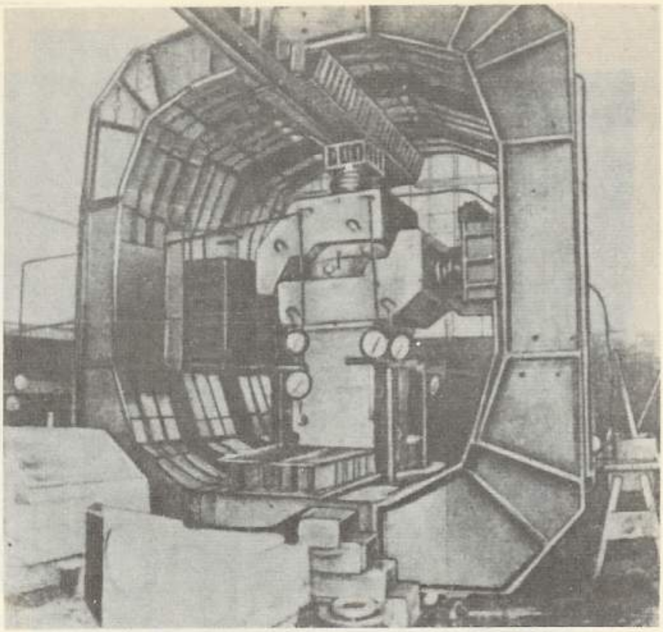


FIGURE 52. Large testing machine for studying the shear resistance of rocky soils

5. In the light of our present state of knowledge of the design of gravity-dam rock foundations, the following formula should be used:

$$\eta = \frac{\Sigma N \tan\psi}{\Sigma E}, \tag{4}$$

where $\tan\psi$ = coefficient of shear $\frac{\tau}{\sigma}$; the other symbols are as previously given.

On the basis of the data collected up to the present, graphs were plotted giving the shear and sliding coefficients for very firm solid rock foundations, as well as for broken rock-foundation soil of various qualities, the lowest having a structure analogous to dry-stone masonry.

6. The initial experiments, carried out with the aid of a special testing machine to study the deformability of rock foundations, showed that the deformation modulus of naturally cracked foundation rock is considerably lower than that of its constituent stone material.

In the construction of the Bratsk HEP gravity dam, as a result of investigations carried out by its designers, the value of $\tan\psi$ was, for the first time in Soviet construction practice, taken as 0.80 ($\eta = 1.05$). This led to a saving of about 400,000 m^3 of concrete.

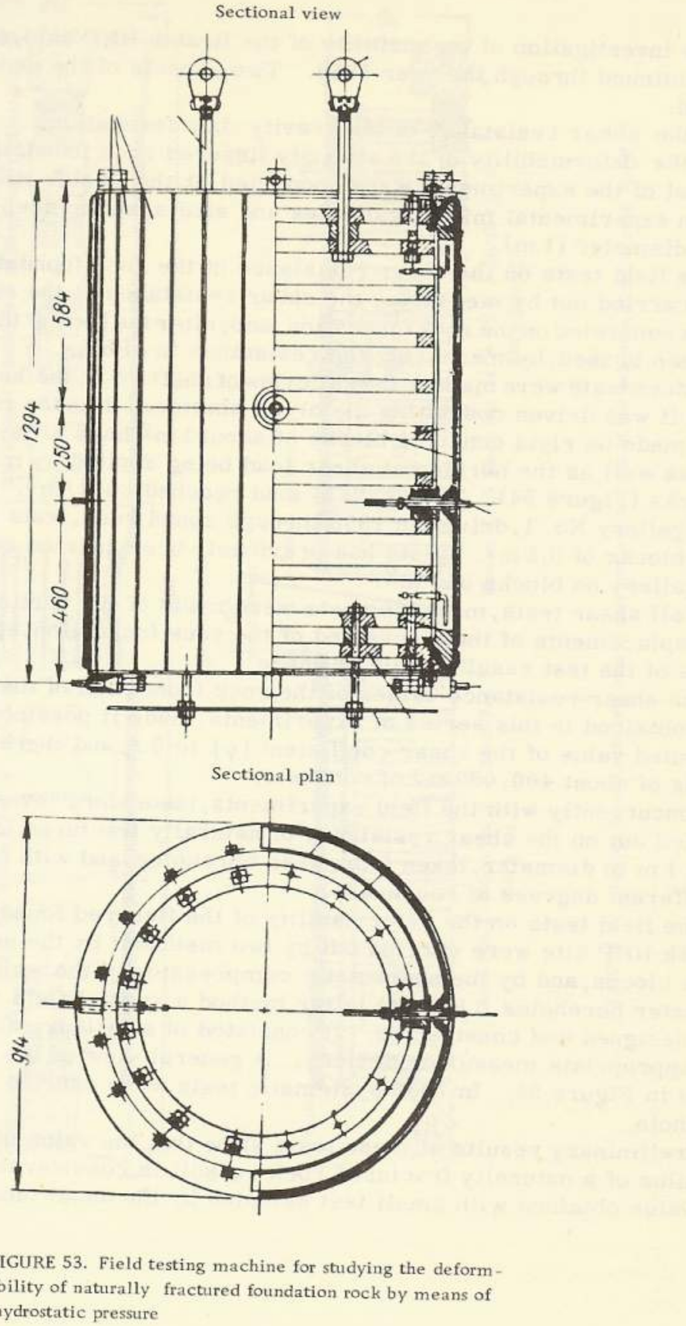


FIGURE 53. Field testing machine for studying the deformability of naturally fractured foundation rock by means of hydrostatic pressure

INVESTIGATIONS ON THE STABILITY OF THE BRATSK HEP CONCRETE DAM ERECTED ON A ROCK FOUNDATION

Responsible for Research: P. D. Evdokimov, Candidate of Technical Sciences, Senior Research Worker

The investigation of the stability of the Bratsk HEP dam, started in 1956 and continued through the year 1958. Two aspects of the problem were studied:

1) the shear resistance of the gravity-dam foundation;

2) the deformability of the strongly fissured rock foundation of the dam.

Most of the experiments were conducted at the Bratsk HEP construction site, in experimental mining galleries and shafts, and in deep boreholes of large diameter (1 m).

The field tests on the shear resistance of the rock foundation of the dam were carried out by measuring the shear resistance at the contact plane of blocks concreted on the rock foundation, and, after the bond at the contact plane had been broken, by measuring the resistance to sliding.

Sixteen tests were made in the galleries of shaft No. 2, the subsoil through which it was driven consisting mainly of shattered diabase rock. The tests were made on rigid concrete blocks of about 1 m^2 base area, the vertical load, as well as the horizontal shear load being applied by means of hydraulic jacks (Figure 54). The vertical load reached $200 t/m^2$.

In gallery No. 1, driven in 1958 through sound rock, tests were made on eight blocks of 0.5 m^2. Tests had previously been carried out in 1956 in this gallery on blocks of 1 m^2.

In all shear tests, measurements were made of the vertical and horizontal displacements of the blocks and of the rock foundation, so that a full analysis of the test results could be made.

The shear-resistance values of the rock foundation of the Bratsk HEP dam, obtained in this series of experiments, made it possible to raise the computed value of the shear coefficient $[\psi]$ to 0.8, and thereby to effect a saving of about 400,000 m^3 of concrete.

Concurrently with the field experiments, laboratory investigations were carried out on the shear resistance of naturally fractured large test samples, up to 1 m in diameter, taken from deep boreholes, and with contact surfaces of different degrees of roughness.

The field tests on the deformability of the fissured foundation rock of the Bratsk HEP site were carried out by two methods: by the use of rigid concrete blocks, and by the hydrostatic compression of the walls of large-diameter boreholes. For the latter method a special field testing machine was designed and constructed. It consisted of a cylindrical hydraulic die and appropriate measuring devices. A general view of the apparatus is given in Figure 55. In 1958 systematic tests were carried out on one borehole.

Preliminary results of these tests show that the value of the deformation modulus of a naturally fractured rock subsoil is considerably lower than the value obtained with small test samples by the usual laboratory methods.

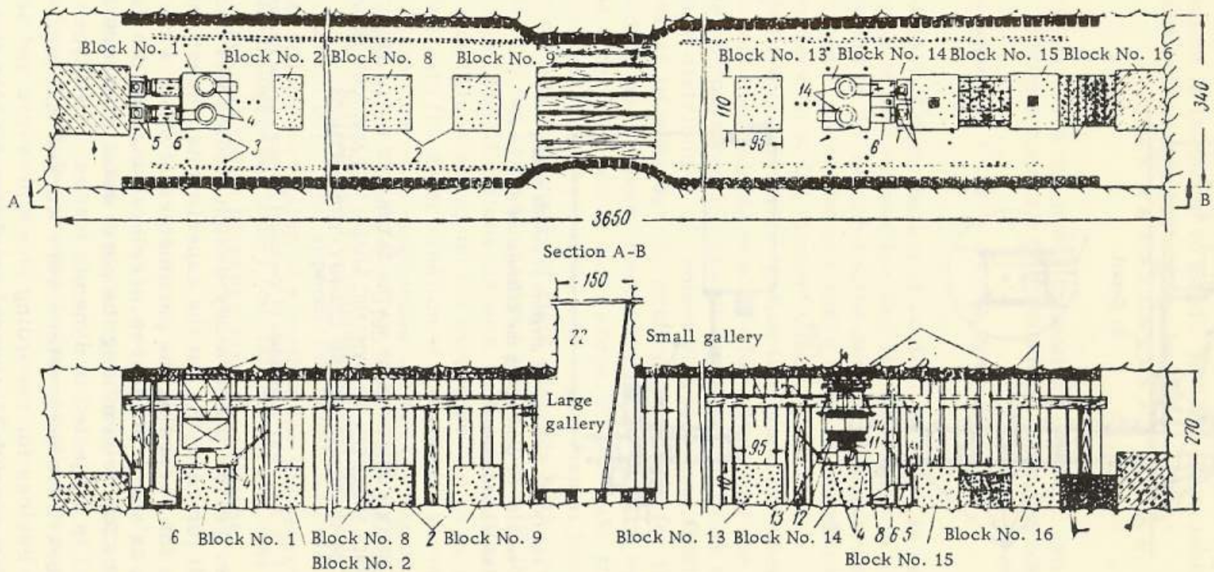


FIGURE 54. Layout of concrete test blocks and grouted-in parts in the galleries of mine shaft No. 2

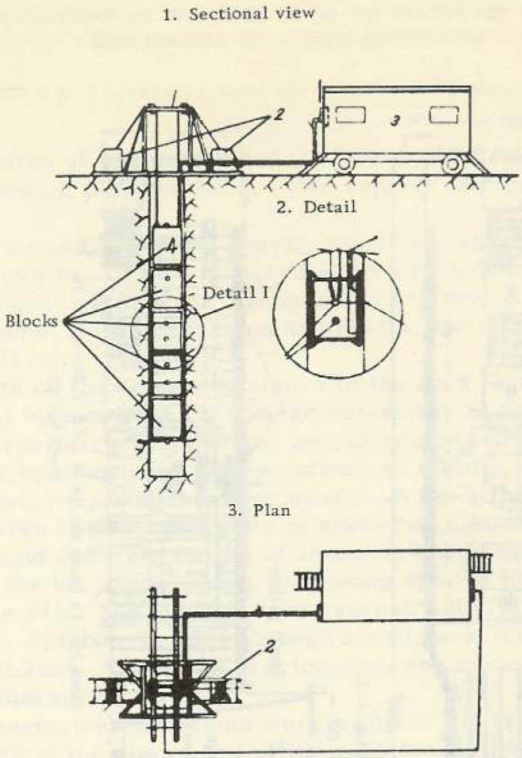


FIGURE 55. Schematic diagram of mobile testing machine for determining the deformation modulus of rock foundations

STUDY OF STABILITY CONDITIONS OF WATER-SATURATED EARTH STRUCTURES AND SUBSOILS SUBJECTED TO DYNAMIC LOADS; WORKING OUT MEANS OF STABILIZATION AND COMPUTATION METHODS

Responsible for Research: L. D. Aptekar', Candidate of Technical Sciences, Senior Research Worker

In the year under review preparatory-design studies were completed and new apparatus built for carrying out the experimental investigation. This included apparatus for recording the parameters of shock waves of strong disturbances, such as shock pressures, accelerations, and displacements, as well as apparatus for determining the pore-water pressure in saturated sands when the soil is subjected to dynamic forces. Two types of recording devices for the shock-wave parameters were worked out: piezoelectric and tensometric. The devices for recording shock pressure and acceleration were provided with a special 24-channel amplifier, as also was the pore-water-pressure recording device. In addition, a special three-channel semiconductor amplifier was provided.

In connection with this same subject, as related to the general investigation program, a laboratory vibration machine was designed and constructed. Its purpose was to study the rheological processes in saturated sandy soils, which develop, as has been demonstrated in the laboratory, when an oscillatory movement is imparted to sand. In 1958 these testing machines were tried out and systematic tests were begun.

DIRECTIVES FOR THE DESIGN OF DAMS FOR STABILITY AND RESISTANCE TO SETTLEMENT, AND STRESS DISTRIBUTION ALONG THE CONTACT PLANES

Responsible for Research: T. F. Lipovetskaya, Staff Engineer

The problem of distribution of contact stresses is one of the most important problems of soil mechanics, and it determines to a considerable extent the dimensions of the structure, the degree of reinforcement, and consequently its cost. Nevertheless, there were no standard directives dealing with this subject up to the present time.

The objective of this investigation therefore has been to study the methods used in current engineering practice for computing the contact stresses; to analyze the comparative designs of separate elements of hydro structures with different distribution of contact stresses; and, finally, on the basis of the above material, to work out the guiding principles for the computation of the stresses acting on the base of concrete dams built on soft (not rocky) subsoils. The directives which were prepared also take into account the results of other existing theoretical and experimental investigations in this field. The directives consist of eight chapters with five appendices.

In addition to the general chapters which deal with the purpose and scope of the technical specifications, as well as the classification of soils, of foundations, and of forces acting on the structure, the work contains the directives dealing with the determination of the contact stresses for concrete foundation slabs laid on homogeneous sandy clay foundations and on heterogeneous foundations. There are also directives dealing with the special features of the construction of foundations and the order in which the work should be carried out. The appendices include calculation tables for rigid foundations, and a description of a method for the determination of contact stresses in homogeneous sandy foundations (in the case of eccentrically loaded foundations) and in heterogeneous foundations.

PRINCIPLES OF DESIGN OF HYDRO STRUCTURES SITUATED IN SEISMIC AREAS

Responsible for Research: B. N. Barshevskii, Candidate of Technical Sciences, Senior Research Worker

The objective of this study was to work out the principles for the design of hydro structures situated in seismic areas.

VNIIG collaborated in this study with the Tbilisi and Leningrad branches of the Gidroenergoproekt. VNIIG collected and analyzed the material (including the published norms) relating to the design of hydro structures in

seismic areas, which also included the work of other scientific research bodies in the field of quakeproof structures. The existing standard reference work on this subject is "Norms and Rules for Construction in Seismic Areas (SN-8-57)"*, published in 1957, which includes a section dealing with hydro structures.

As a result of a critical review and analysis of the available material on seismological calculations in the designing of hydro structures collected for this research, it was concluded that some corrections should be made in the section of SN-8-57 specifications dealing with hydro structures.

The report consists of two parts: Part I deals with the reasons for the recommended corrections and additions to the section "Hydro structures" of SN-8-57 specifications. The following are the main changes proposed:

a) The addition of tables with the help of which it will be possible to define more precisely the earthquake intensity number in relation to local soil conditions.

b) It is shown that, in most cases, the coefficient related to the dynamic characteristics of a hydro structure can be taken to be a constant.

c) Changes are introduced in the formulas recommended by SN-8-57, which take into account the dynamic water pressure on the retaining structures.

d) It is proposed not to adopt the recommendation contained in SN-8-57 to check the stability of dams along their construction joints.

e) It is proposed to carry out experiments to ascertain the dynamic stability of saturated sand in hydraulic-fill dams.

Part II gives the revised version of the section "Hydro-engineering structures" of SN-8-57, incorporating the above amendments.

VNIIG im. B. E. VEDENEEV LABORATORY FOR PHYSICOCHEMICAL RESEARCH AND SOIL ANALYSIS

Head: B. F. Rel'tov, Candidate of Technical Sciences, Senior Research Worker

TEST OF THE RADIOMETRIC METHOD DEVISED BY VNIIG FOR CHECKING SOIL DENSITIES UNDER FIELD CONDITIONS

Responsible for Research: Yu. S. Bol'shakova, Candidate of Chemical Sciences, Junior Research Worker
Research by: G. A. Kraev, Engineer

The objective of the study was to test the radiometric method for checking soil densities under field conditions.

The work was divided into two parts. First, experiments were made on the absorption of a wide gamma-ray beam by soils of different moisture content and density, as well as for different depths of irradiation.

From the results obtained it was possible to establish the optimum distance at which the effective mass-absorption (attenuation) coefficient of a wide beam remains practically constant while the density varies.

$$\left.\frac{\Delta \mu'_{\text{eff. w.}}}{\Delta \rho} = 0\right|_{r=\text{const}} .$$

* [Normy i pravila stroitel'stva v seismicheskikh raionakh (CN-8-57).]

The second part of the study consisted in the design, construction, and testing of a deep radiometric probe for determining soil densities. This part of the work was done in collaboration with Ukrgidep.

During the work it became necessary to simplify some of the subassemblies of the probe, and a simplified model was worked out, which retains the basic principles of the radiometric probe devised by VNIIG.

The tests with the probe showed that its mechanical performance is satisfactory, and that no constructional changes are required. As regards the radiometric measuring device, however, it was found necessary to make a number of changes in the construction of the gamma-quanta counters. It was also found that the standard counter (type B) is unsuitable for practical work under field conditions; not only is it bulky, but the readings are materially affected by the electric-field induction, by fluctuations in the feeding voltage, and by the ambient moisture. It is therefore advisable to use special apparatus suitable for field conditions.

VNIIG IMENI B. E. VEDENEEV ENGINEERING-GEOLOGY LABORATORY

Head: A. A. Glaz', Candidate of Technical Sciences, Senior Research Worker

INVESTIGATION OF LOESS AS FOUNDATION AND BUILDING MATERIAL FOR HYDRO STRUCTURES

Responsible for Research: A. A. Glaz', Candidate of Technical Sciences, Senior Research Worker

The purpose of this study was to work out methods for the investigation and the computation of the settlement of macroporous loess foundations of water-retaining hydro structures.

The 1958 program included:

a) the study of observational data on the settlement of several hydro structures built on a loess foundation;

b) the planning and carrying out of investigations on the deformability and strength of loess soils of various genetic types;

c) draft directives for the determination of the deformability characteristics of loess soils in hydro structures.

The study of the experimental data and of the observations on the settlement of hydro structures of the Stalinabad HEP-2, Farkhad HEP and Nizhne-Bos-Sui HEP-4 showed that settlement was caused by the gradual destruction of the salts which act as a binder in the loess foundation.

The examination of the records of a number of research projects, and the study of the results of observations extending over many years at the above hydro plants, as well as at the forebay of the Tavak HEP which belongs to the Chirchik River cascade system, showed that, generally, the actual settlement of the hydro structures on a loess subsoil does not correspond to the calculated values.

This discrepancy arises because the present methods of computation do not take into account the effect of seepage on the gradual destruction of the salt cementing-material in the loess soil.

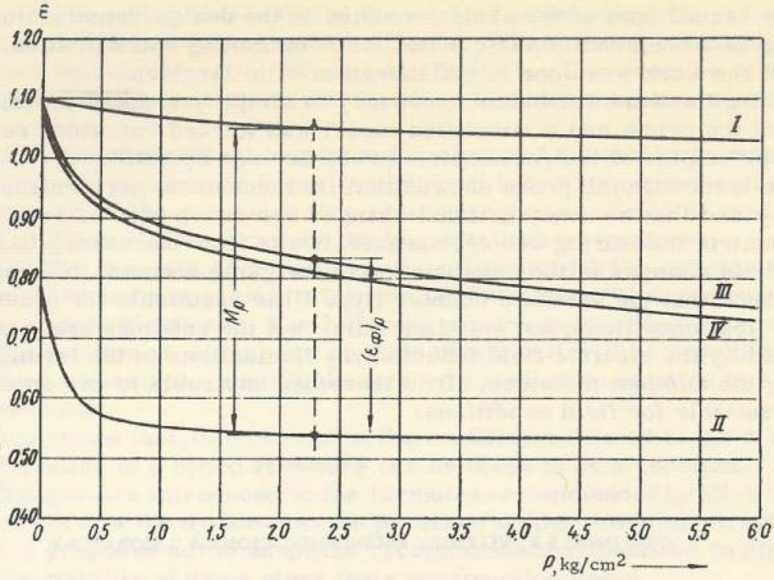


FIGURE 56. Loess soil at the Perepadnaya HEP(Tadzhik S. S. R.)

I–compression curve of an undisturbed loess sample under natural moisture conditions; II–compression curve of a disturbed saturated loess sample (test made under water); III–compression curve of a monolithic wetted loess sample under zero-load conditions; IV–compression curve under seepage conditions.

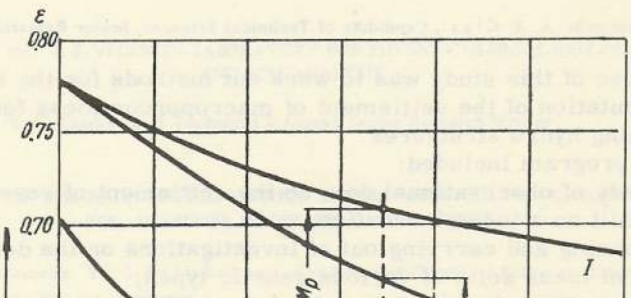


FIGURE 57. Loess soil at the Dneprodzerzhinsk HEP

I–compression curve of a monolithic loess sample under natural moisture conditions; II–compression curve of a disturbed saturated loess sample (test made under water); III–compression curve of a monolithic wetted loess sample under zero-load conditions.

The experimental investigation established that for confined loess soils subjected to a given load under conditions of wetting and prolonged seepage, the maximum possible settlement is determined by its total macroporosity (M_p).

The post-subsidence* deformation of macroporous soils can considerably exceed the subsidence, and is determined by the value of the residual macroporosity $(\varepsilon_{av})_r$, being the difference between the total macroporosity and the macroporosity which is destroyed by wetting under the given load conditions.

Figures 56 and 57 show the results of tests on loess samples which indicate a possible relationship between the subsidence and subsequent settlement. The samples were taken from the Vakhsh River area in the Tadzhik S.S.R. (Figure 56), and from Dneprodzerzhinsk in the Ukrainian S.S.R. (Figure 57).

The method proposed here for determining the values of the total macroporosity, as well as those of the macroporosity which is destroyed by wetting and seepage, may be used to evaluate the subsidence and the post-subsidence settlement when designing hydro structures built on a macroporous loess subsoil

INVESTIGATION OF THE PHYSICOMECHANICAL AND ENGINEERING PROPERTIES OF FROZEN SOILS AS SUBSOILS FOR HYDRO STRUCTURES

Responsible for Research: A. K. Blatt, Candidate of Geological and Mineralogical Sciences

The aim of this project was to lay the groundwork for establishing an experimental laboratory in VNIIG for the investigation of the physicomechanical properties of frozen soils as subsoils for hydro structures. The investigations dealt with:

1) study of the latest technical literature on the subject of the physicomechanical properties of frozen soils;

2) definition of the most immediate problems on frozen soils requiring experimental investigation;

3) setting up a list of the equipment required for the experimental laboratory;

4) planning the layout of the laboratory.

A review of the present state of knowledge of the mechanophysical properties during the thawing-out process have not been sufficiently developed, and require improvement. The following basic problems were singled out for further investigation:

1) study of the influence of the freezing and thawing processes on the mechanophysical properties of cohesive soils;

2) study of compressibility and shear resistance of frozen soils during the thawing-out process;

3) study of the increase in the shear resistance of thawing soil, as a function of the load and the degree of consolidation.

As the first steps for carrying out these investigations, a list of the necessary equipment was drawn up and the laboratory layout was planned.

* ["Subsidence" is used for the Russian term "prosadka", which is the settlement of loess soils when loaded and wetted.]

INVESTIGATION OF THE SETTLEMENT OF THE FOREBAY STRUCTURES OF THE FARKHAD HEP

(Technical aid to the Uzbekenergo)

Responsible for Research: A. A. Glaz', Candidate of Technical Sciences, Senior Research Worker
Research by: L. A. Morozova, Senior Engineer

The aim of this investigation was to obtain more precise information on the causes of the deformations observed in the hydro structures of the Farkhad HEP, and then to work out appropriate remedial measures.

In the course of the investigation, field observations were made of the settlements, of the displacements of various parts of the structures, and of the seepage. The loesslike foundation was tested in the laboratory, and the soil water was analyzed to ascertain its mineral composition. In addition, recommendations were worked out for the reconstruction of the joints of the structures.

The results were as follows:

1. The rate of settlement of parts of the structure in the period 1957 – 1958 was smaller than that observed in the period 1955 – 1956. For example, the rate of settlement of bench marks made along the south regulator, on the left and right wings of the forebay, was reduced to one third, and that of bench marks along sections in line with the upper service bridge, to one tenth of the 1955 – 1956 values. There was no change in the rate of settlement of bench marks in line with the cut-off wall.

2. The measurements of the deviation of the forebay front wall, carried out by VNIIG by means of visual sighting, with a geodetic rectangle and by means of invar wire, showed that these deviations were considerable, both on the headwater and on the tailwater side.

3. Sighting along the derrick-shaped abutments did not reveal any change in the slope of the pipelines.

4. The water level in the piezometric tubes was rising continuously and had reached 2.5 m.

5. The seepage line in the forebay foundation rose by an average of one meter during the same period.

6. The discharge through the upper drainage pipes doubled, and the discharge through the drainage system below the penstock increased 2.5 times.

7. The sharp rise in the water level (to 2.5 m) led to the increase of content of easily soluble compounds of NaCl and Na_2SO_4 in the soil water, apparently due to the water coming into contact with new soil layers rich in these salts.

The results of the investigations carried out in 1958 conform to the view previously expressed by VNIIG regarding the causes of the deformations observed over a long period at the forebay hydro structures of the Farkhad HEP.

In view of the continuous oscillatory movement of the front wall, and the continuous rise of the water level in the piezometers, VNIIG recommended the following steps to the operating personnel of the Farkhad HEP:

1) to continue observations of the operation of the structures in accordance with the program worked out by VNIIG;

2) to take all possible measures against leakage through the borders and the bed of the forebay structures of the plant;

3) to carry out in 1959 the reconstruction of the damaged joints;

4) to refer to VNIIG, for the purpose of analysis and for drawing general conclusions, the results of the observations and the reports on the implementation of the Institute's recommendations.

STUDY OF THE PHYSICOMECHANICAL PROPERTIES OF LOESS SOILS IN THE JUNCTION BETWEEN THE EARTH DAM AND THE RIGHT BANK AT THE DNEPRODZERZHINSK HEP

Responsible for Research: L. A. Morozova, Senior Engineer

The aim of the investigation was to study the deformability, strength and permeability of loesslike soils under working conditions of a hydro structure, in connection with the design of the junction between the earth dam and the right bank of the Dnieper River.

The investigation was carried out in accordance with methods developed in the Engineering-Geology Laboratory of VNIIG.

The results of the investigation were as follows:

1. The loess-type deposits consist of powderlike sandy and clayey loams*, composed mainly of quartz. Their clay fraction contains kaolins and hydromica. Their content of soluble compounds varies in amount from 2% to 17%, and consists mainly of sparingly soluble calcium and magnesium carbonates.

2. In its natural state the soil has the following characteristics: a low moisture content; a high porosity (45% – 49%); a low compressibility; high shear resistance ($\varphi \cong 35 - 30^\circ$; $C \cong 0.90 - 0.30\,\mathrm{kg/cm^2}$).

3. Loesslike soils become less porous with wetting, and the porosity is further reduced as the applied load is increased. Wetting does not, however, exclude the possibility of further settlement under conditions of seepage.

4. Seepage conditions bring about an additional settlement which may be accompanied by the total destruction of the structural salt binders. Thus, for the most characteristic varieties of loesslike soils, the value of the post-subsidence settlement amounted to 170% of the subsidence, at a load of $P = 3\,\mathrm{kg/cm^2}$. The settlement characteristics of loesslike soils under different conditions are illustrated in Figure 58.

On the basis of this investigation the following recommendations were made:

1. That the slope at the junction of the earth dam with the right bank of the Dnieper River should have a longitudinal inclination increasing from 1: 1.5 to 1: 3.

2. That the slope of the right river bank be blanketed with a 50 cm thick layer of blended loesslike clay loam extending to 500 – 600 m from the junction as a protection from the activity of the reservoir water on this part of the bank.

* ["Sandy loam" is used for the Russian term "supes'", which, according to Soviet soil classification, contains 3 – 10% by weight of clayey particles (<0.005 mm); "clayey loam" is used for the Russian term "suglinok", which contains 10 – 30% of clayey particles.]

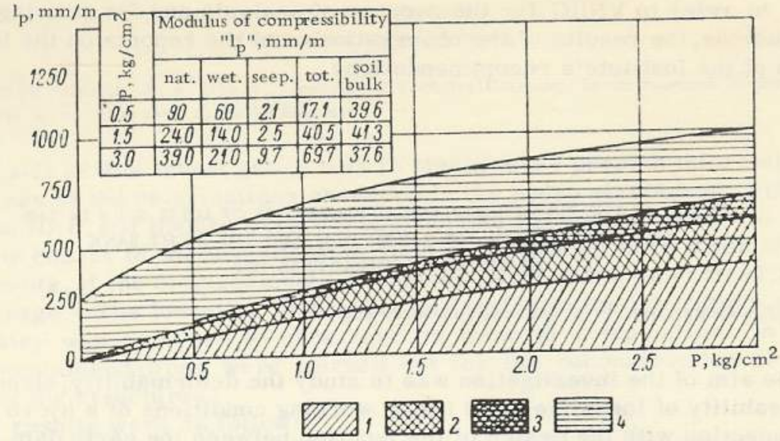


FIGURE 58. Curves of mean compressibility coefficients for loesslike clay loam

3. That the gorge should be blocked with high-quality fill.

4. In view of the unavoidable decrease in the strength of loessial soil due to wetting and seepage, the designers were advised to check the stability of the bank slopes in the submerged zone, and also to calculate the expected settlement of the dam and of the bank slopes in the vicinity of the junction, taking into account the minimum values of the shear-resistance characteristics, and making use of the data obtained in the VNIIG investigations.

The above recommendations were utilized in the design.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR THE STUDY OF ICE-THERMAL CONDITIONS

Head: Professor A. M. Estifeev, Candidate of Technical Sciences, Senior Research Worker

PRINCIPLES OF HYDRO-STRUCTURE DESIGN UNDER PERMAFROST CONDITIONS
(Thermotechnical Problems)

Responsible for Research: G. S. Shadrin, Candidate of Technical Sciences, Senior Research Worker.

The aim of the investigation was to work out methods for the thermotechnical computations used in the design of hydroengineering structures built in permafrost conditions.

Under such conditions all engineering structures, and especially hydro structures, disturb the thermal conditions of the foundation. When the soil thaws out, its structural properties deteriorate; settlement and seepage may damage the structure and even put it out of operation.

The report describes the methods used for the thermotechnical computations in the design of hydro structures. It can be divided into two sections, according to the subject matter:

1. General section (Chapters I, II, IV) deals with the analytical, graphical-analytical and other methods for the solution of thermal problems, including the methods worked out in the ice-thermal laboratory of the VNIIG (method of graphic representation of a stationary thermal field, graphical-analytical method for solving the Stephan problem), which can be used in the thermal computations for hydro structures. This section also gives the values of the thermal-physical constants of soils.

2. Experimental section (Chapters III, V, VII) deals with the development of the method of model analysis, recommended for the solution of complicated thermal problems, and especially for the comparison of the thermal state in earth dams in whose design VNIIG collaborated, and which are completed and already in operation. In addition, this section gives designs of dams forming part of hydro power plants, and deals also with work done abroad based on the material of the Permafrost Institute of the Academy of Sciences of the U.S.S.R. and several other sources.

As a result of this work, an evaluation is made of the feasibility of various methods, including experimental methods, for use in solving thermal problems in the computation of hydro structures. The method of model analysis is recommended in cases of inhomogeneous soils and complicated limiting conditions, and particularly for the tridimensional problem. The method is based on the comparison of the thermal state of the dam with that of its model.

SCIENTIFIC RESEARCH STATION OF THE HYDRO PROJECT IMENI S. Ya. ZHUK
SECTION FOR THE INVESTIGATION OF SUBSOILS AND EARTH STRUCTURES

Head: M. D. Dundukov, Engineer

STUDY OF THIXOTROPIC AND QUICKSAND PROPERTIES OF FOUNDATIONS OF HYDROELECTRIC POWER PLANTS

Research by: V. V. Radina, Engineer

In the course of the geologic investigation of the foundation soil of the Lower-Ob' HEP, when drilling through silty differing soil of the Salemal' series, indications were found pointing to a low structural strength of these soils (thixotropy and quicksand properties). It was necessary to make a special investigation of the physicomechanical properties of such soils, so as to determine the conditions under which they lose their structural strength when subjected to vertical and horizontal forces, as well as to vibrating loads.

The investigation showed that the HEP foundation soil was characterized by a pronounced silty texture (the number of silt-sized particles ~ 50%), a low colloid content (1 – 2.5%) with a relatively small clay fraction, and a weathered

condition of the quartz and feldspar grains, which were coated with a film of ferrous hydroxide.

The surface of the quartz grains was uneven, as if corroded, the faces being cracked in various directions. The feldspar grains were sericitized. The content in organic (carbon) matter ranged from a few hundredths to 1.4 %. Soluble salts made up 0.003%. Easily decomposed compounds of iron and amorphous silica were also present. The soil is a reducing medium. The soil water is slightly alkaline (pH = 7.41 - 7.53).

The microbiological investigation of the soil samples revealed the presence of silicate bacteria, which are capable of destroying potassium alumosilicates, apatites, and phosphorites, forming a bluish slime consisting of amorphous silicic acid and hydroxides.

According to their mineral composition, the sand and powder sizes belong to the quartz feldspars. Their grains are angular and partly rounded.

The clay fraction is composed mainly of hydromica, but montmorillonite and quartz are also present. The thixotropy of the soils for the maximum shear resistance, appears beginning with a soil consistency corresponding to the plastic limit ($w > w_p$), while the flowability appears beginning with the liquid limit ($w \gg w_l$).

At natural density and moisture values, no thixotropy or quicksand properties are observed. With a water content exceeding the liquid limit, the density of the paste increases with time.

There are two stages in the strengthening process of the soil structure: in the first stage the weight of the soil reduces the distance between the particles; in the second stage the development of a structural bond between the particles further strengthens the soil structure. The more dispersed* the soil and the richer it is in colloids, the shorter the first stage becomes, even to the point of being completely eliminated.

According to Lebedev, the increase in density up to complete saturation, independent of the amount of the initial water content, is observed only in sand whose relative density may reach the value $D = 0.6 - 0.7$. The corresponding increase in strength is reflected in the rise of the shear resistance by a factor of 1.5 to 2.5.

The absolute value of the structural strength of the soil paste ranges from only a few grams up to as much as 165 g/cm^2, according to the nature of the soil

The silty soils tested were found to have fairly high shear-resistance values both at natural density and also under all test conditions; with these soils the loosening occurring with water saturation caused a considerable drop in shear resistance. Experiments made on the effects of prolonged shear on these silty soils show that they undergo plastic deformation; the looser the soil, the smaller the shear load required to produce this effect.

In sand, shear resistance is lowered by vibration. For a given density, its stability depends on the vibration frequency and on the load. The sand density is not increased by vibration.

Drainage conditions were only observed to affect the soil strength in samples with densities at which pore-water pressure in the soil was possible.

* [The term "dispersed soils" used in the Soviet literature is defined as "loose and plastic soils composed of particles of different sizes".]

The compressibility of sandy loams of semihard consistency* at $P = 8\,kg/cm^2$ is characterized by a residual settlement equal to 2-3%, as against 7-9% for soft plastic sandy loams.

Tests made after the destruction and removal of the gel-like colloidal film adsorbed on the surface of the particles of noncohesive soils, showed that their thixotropic and quicksand properties had disappeared. Similar results were obtained by Rossel' in the case of clays (Freindlikh, "Tiksotropiya" (Thixotropy) 1935).

Special tests were conducted to clarify the function of the silicate bacteria and the increase in the colloid content of sands. Although the tests had not been completed at the time of writing, they have provided enough information to confirm the view that these factors contribute to the formation of true quicksands.

In conclusion, it should be noted that the basic feature of the investigated soils is their marked silty texture.

When these soils are in a loose and saturated state, the presence of mineral and organic colloids imparts to them the character of true quicksands.

In the study of such soils it is important to determine the quality and the quantity of the colloids present, as well as the values of moisture content and density at which the soil develops thixotropic and quicksand properties. It is particularly important to ascertain the shear load, expressed as a fraction of the normal load, at which plastic flow can develop.

The soils investigated proved to be suitable for foundations, provided that their natural structure remained undisturbed, and that steps are taken to prevent loosening.

The results of the investigations were utilized in the design of the Salekhard hydro development.

INVESTIGATION OF THE STABILITY OF AN EARTH-DAM SLOPE

Heads of Research: N. V. Zamorina, Engineer
E. M. Shvei, Engineer

Research Team: I. V. Dudler, Engineer
V. L. Popova, Engineer
M. G. Korshunov, Engineer

The aim of the investigation was to evaluate the stability of the slope of the earth dam at the Stalingrad HEP. The basic problem was to ascertain experimentally the conditions under which sand subjected to dynamic influences starts to flow, to lay down the requirements regarding the density of the fill material for the body of the dam built of local sands, and to reconsider whether it was necessary, as specified in the technical design, to add to the fill coarse-grained sand.

The investigation was made at the Stalingradgidrostroi under close-to-field conditions. The sand intended for the construction of the dam extended

* [Relative consistency A = 0.75 – 1.]

for 450 m along the bank of the Volga River, and an experimental sand block about 7 m high with a submerged slope of 1:6 was built up by the hydraulic-fill method.

The investigation on this experimental block included blasting, which was considered to be the most effective form of dynamic action for testing, being comparable to those liable to occur during the actual operation of the hydro structures.

Most of the experiments were carried out on the portion where the density of the sand approximated that usual for hydraulic fill. The unit dry weight of the soil was $1.55\,t/m^3$, the porosity 42%, and the residual density 0.26. The critical density of these sands was higher, and corresponded to a unit dry weight of $1.58\,t/m^3$.

In one part of the block the sand was compacted by pneumatic pulsation, according to a method developed by the scientific research department of Orgenergostroi. The unit dry weight and the porosity after compaction, up to depths exceeding 4 m, were respectively $1.60\,t/m^3$ and 40%.

In another part of the test block the slope was loaded with a layer of coarse-grained sand whose thickness (2 m) was approximately equal to half the depth of the placed charges. The object of this measure was to replace the upper layer which, due to the absence of a load from a covering layer, was the least stable portion, by a material such as coarse-grained sand, which is more resistant to flow conditions.

These investigations were a continuation of earlier work done by the scientific research department of the Kuibyshevgidrostroi, when methods were worked out for the evaluation of the flow conditions of sand. These methods included visual observations supplemented by measurements and sketches; measurement of the settlements and displacements of the slope, both above and below water; measurement of the settlement of "sunken" bench marks which indicate the loss of the bearing properties of the sand at various depths, the rate of sinking of the bench marks being recorded by a cinecamera; measurement of the excess pore-water pressure, with the aid of tension indicators and piezometers. For determining the density and the structural strength of the sand, the sounding method was widely used, and it provided a reliable criterion for judging if the sand was liable to flow.

The results of the investigations confirmed conclusions arrived at in earlier work, including the conclusion that the occurrence of flow conditions for sand is related mainly to its density and structural strength. Values were given for the empirical coefficients of the previously proposed formulas, determining the dimensions of the zone of flow conditions of the submerged hydraulic fill consisting of local Stalingrad quarry sands, in relation to the values of the dynamic influences.

It was established that in order to achieve dynamic stability of the slope of the Stalingrad HEP dam built of local sands, it is essential that the weight of a soil matrix with a safety factor of 50% be $\geqslant 1.58\,t/m^3$, and that when using the dynamic-sounding method, not less than seven impacts are required on the average per 10 cm of penetration. It was further established that a layer of coarse-grained sand laid on the submerged slope, whose thickness is of the order of half the depth of the laid charges, does not affect the stability of the sand mass when the charge is detonated.

The results obtained were used in drawing up the technical specifications for the hydraulic fill of the Stalingrad HEP dam, making it possible to reduce

the earlier requirements regarding the unit weight of the sand, and to avoid the need to add coarse-grained sand to the fill sources.

LABORATORY METHODS FOR INVESTIGATING THE PHYSICOMECHANICAL PROPERTIES OF ROCKY SOILS IN HYDRO CONSTRUCTION

Research Team: B. P. Belikov, Candidate of Geological and Mineralogical Sciences
I. N. Gudaev, Engineer
Professor B. V. Zalesskii
Yu. A. Rosanov, Candidate of Geological and Mineralogical Sciences

These directives are intended for the staff of the laboratories of the geological expeditions of the Scientific Research Institute of the Gidroproekt im. S.Ya Zhuk. They serve as a guide for the laboratory investigation of the properties of rock intended for use as foundations, concrete aggregate, and building stone.

These directives are based on existing technical conditions, and make use of the work of the Institute for the Geology of Mineral and Ores, Petrography, Mineralogy and Geochemistry of the Academy of Sciences of the U.S.S.R., the scientific research division of the geological research department of Gidroproekt, the VNIIG, as well as a number of other research and construction institutions.

The directives consist of three parts. Part I contains basic technical instructions for taking samples used in the determination of the physicomechanical properties of rock and for special geological and petrographical investigations. Part II gives the methods for determining the physical properties of rocks, and Part III the methods for determining their mechanical properties.

THE BASIC PHYSICAL LAWS GOVERNING PORE-WATER PRESSURE DURING THE CONSOLIDATION (COMPACTION) OF CLAY SOILS

Head of Research: Ya. L. Kogan, Candidate of Geological and Mineralogical Sciences
Research by: E. K. Alekseeva, Engineer

The present methods of determining the pore-water pressure developing in dispersed clay soils, when their state of stress is changed, are based largely on hypotheses not sufficiently confirmed by laboratory and field investigations. When computing structure stability, the pore-water pressure is often presumed to attain its maximum possible value, i.e. a value equal to the additional load due to the weight of the structure. This leads to a considerably higher stability coefficient, and consequently to an increase in the cost of the structure.

The investigations of the physical laws governing the dynamics of the development of pore-water pressure during the consolidation process of dispersed clay soil, permit a more precise computation of the

stability and the settlement of hydro structures. The present investigations form part of the wider subject of the study of the mechanical properties of soils, undertaken by the scientific research section of the Gidroproekt.

Theoretical analysis and a review of the technical literature led to the conclusion that, in the consolidation process, the distribution of the pressure of the external load between the soil structural matrix and the pore water depends mainly on the gas content of the pore-water, the soil density, the soil dispersion characterizing the chemically bound water; and the drainage conditions.

At full saturation of a soil, the pore-water pressure at the moment of application of external load is equal to the load, and under closed-system conditions it remains unchanged for any desired length of time. This proposition of Terzagy-Gersevanov can be experimentally demonstrated.

If entrapped air is present in the soil, the external load is transmitted at the moment of its application to the chemically bound water surrounding the soil particles. The pore-water pressure rises gradually with the compression of the entrapped-air bubbles which are located in the free pore water.

The report describes a mechanical model of a three-phase soil, used for the study of the consolidation process and of the distribution of the pressure resulting from the external load. Details are given of the specially designed apparatus used in the investigation, and the report presents the results of the experiments carried out in the flow gages and on the special test bench built in connection with the model analysis of the linear consolidation problem.

The experiments showed that, when computing the soil consolidation, the seepage properties of the soil become insignificant as the soil density increases.

MODEL AND FIELD INVESTIGATIONS OF THE MECHANICAL PROPERTIES OF ROCKY SOILS

Research by: B. D. Zelenskii, Engineer

It is only recently that field investigations were started to determine the mechanical properties of rocky soils in their natural occurrence. Special methods were worked out for each particular case, and we can now distinguish the characteristic features of the different methods and experimental techniques.

The aim of this work was to arrive at general conclusions on the basis of the available material, and to recommend the most effective methods. The report will contain five parts, each dealing with a different method, namely:

1. The use of "dies" or of the "elastic half-space" method.
2. The application of a hydrostatic load to the walls of a mining gallery having a circular cross section.
3. The application of a hydrostatic load to the walls of plane fissures.
4. Seismic prospecting.
5. Model testing.

The fifth method is supplementary to the others, since it is not directly concerned with a certain method of field investigation, but with the analysis and the evaluation of the field data.

Part I, dealing with the method of elastic half-space, was completed in 1958. In this method rigid dies are pressed against the floor or the walls of a horizontal mining gallery, and measurements are taken, in relation to a relatively immovable frame, of the deformation of the die base and the displacement of the rock surface adjacent to the die. The load is applied by hydraulic jacks, and the deformations measured by strain gages. The deformation modulus of rock is calculated using the formulas of the theory of elasticity, or empirical formulas obtained by laboratory experiments on models of the field installation.

Part I also contains instructions regarding the choice of the shape and dimensions of the horizontal gallery, the geological aspects of the experiment, and the technique of applying the loads and measuring the deformations. Recommendations are also given regarding the investigation program. The last chapter contains formulas for the analytical treatment of the experimental results. An appendix gives three examples of field investigations of the deformation properties of rock, carried out in Germany, Switzerland, and the U.S.S.R.

FIELD OBSERVATIONS AND INVESTIGATIONS OF THE DEFORMATION OF THE FOUNDATION SUBSOIL OF THE VOLGA HEP IMENI V.I. LENIN AS MEASURED ON 1 JANUARY 1958

Head of Research: S. N. Knyazev, Engineer

Research Team: A. K. Efimov, Engineer
O. A. Berestetskii, Engineer
V. L. Popova, Engineer

The report gives a short description of the type of the structure and of the engineering-geological conditions of the Kuibyshev HEP foundation, with data of: the decrease of the foundation density observed during the excavation of the construction trench; the settlement of different layers of the foundation rock and its total settlement; the inclination and deflection of the construction sections and also their horizontal displacement.

The foundation of the Kuibyshev HEP consists of steeply dipping strata of quaternary and tertiary clays, the sections under construction on the right bank being situated partly above the quaternary and partly above the tertiary deposits.

The excavation of the construction pit was carried out at depths of up to 45 m, with a corresponding lowering of the water table.

The observations were made by geodetic surveying methods. The leveling of the bench marks was accurate to within ± 5 mm, with occasional deviations of up to ± 10 mm.

The rise of the foundation bed observed on excavating the construction pit is estimated at being 200 – 250 mm. As measured on 1 January 1958 the settlement of the HEP section had nearly ceased, after having reached 142 – 230 mm under loads of up to 5 kg/cm^2. On the basis of these field observations, it can be expected that the settlement will completely cease within 3 to 4 years and that the final amount of settlement then reached will be about

200 – 300 mm. Initially the inclination of sections of the construction on the tailwater side reached 0.4 mm/running meter, and on the headwater side 0.1 mm/running meter. Inclination toward the headwater side began in all sections from the moment of load application, this inclination at the end of 1957 reaching 0.3 – 0.43 mm/running meter, with maximum deflection occurring on the slab of section No. 2 where it reached 30 mm.

The horizontal displacements of the sections of the hydroelectric plant were insignificant, and were within the limits of probable observation errors.

These field observations of the deformations in the foundation soil of the Volga HEP were used to solve a number of structural problems in the course of the construction of the hydroelectric plant and the installation of the plant units, particularly the problem of the inclination of their center lines.

CHANGES IN THE PHYSICOMECHANICAL PROPERTIES OF MAIKOP CLAYS AS A RESULT OF INTERMITTENT WETTING

Head: Ya. L. Kogan, Candidate of Geological and Mineralogical Sciences

Research Team: E. K. Alekseeva, Engineer
M. I. Il'yanina, Engineer

The report presents the results of the investigations on the effect of alternate wetting and drying on the physical properties and strength of Maikop clays along the canal path and in the foundation of the hydro structures of the Kuban' – Kalauss channel system.

The thickness of the Maikop-clay layer at its outcrop along the canal is estimated at 6 – 8 m. The layer is characterized by its fissured state and consists of shaly laminas not bound to each other, covered with an iron oxide crust which gives the soil interior a brown color, unlike the dark-gray color usually found in Maikop clays.

The Maikop-clay eluvium structure is inhomogeneous along its depth, both as a result of inclusions of iron compounds and of the degree of weathering. The moisture content ranges between 20.5 and 28.0 %, the density, between 1.85 and 2.04 g/cm^3, the density of the solid constituents between 2.70 and 2.94 g/cm^3, and the plasticity index between 21 and 35 %.

The investigations were carried out in a test pit on the canal path in the Barsuchki region and on soil samples in the laboratory. It was established that when moisture conditions are stable, the Maikop clays and the structural components of their eluvia retain fairly well their compactness and high strength. When this type of clay is subjected to alternate wetting and drying, or alternate freezing and thawing in water, it quickly shows intensive weathering and turns into an umbricated mass of thin lamellas which eventually become a structureless friable mass with very low strength characteristics. The moisture contents rise up to 60 – 80%.

The shear resistance of clay eluvium of approximately natural density and water content, whose structure has been destroyed, is 30% greater than

that of clays that have preserved their structure. This can be explained by the fact that a fissured condition is the natural state of the eluvium.

The shear-resistance values along the contact planes of the soil laminas on which the iron oxide encrustation is not removed are $\tan\varphi = 0.17 - 0.32$ and $C = 0.04 - 0.28\ \text{kg/cm}^2$. If the crust is removed, the corresponding values are $\tan\varphi = 0.13 - 0.16$ and $C = 0.06 - 0.15\ \text{kg/cm}^2$.

The swelling of the Maikop-clay eluvia under a load of $1\ \text{kg/cm}^2$ reaches 2 to 10%. When clay samples are alternately wetted and dried, the amplitude of the volume changes is 16 to 23% for loads of $1\ \text{kg/cm}^2$, and 10 to 18% for loads of $2\ \text{kg/cm}^2$.

The results of these investigations showed that it is essential to take measures to prevent intermittent wetting or freezing of the Maikop clays forming the canal slopes, or, alternatively, to replace them by more stable soils in all portions of the canal slopes liable to be exposed to these conditions.

SETTLEMENTS AND HORIZONTAL DISPLACEMENTS AT THE CONCRETE SPILLWAY DAM OF THE VOLGA HEP IMENI V. I. LENIN ACCORDING TO FIELD-OBSERVATION DATA ON 1 JANUARY 1958

Head of Research: S. N. Knyazev, Engineer

Research Team: A. K. Efimov, Engineer
O. A. Berestetskii, Engineer

The report gives a short description of the structure and of its foundation, explains the methods applied in the investigation, and evaluates the accuracy of the control observations of the settlement, the inclination, and the horizontal displacements of the structure as from the time of construction. In addition, the report analyzes the observational data and forecasts the future settlement, as well as the magnitude and the increase in the loads.

The foundation of the dam consists of alluvial sands to a depth of 60 – 70 m, below which is a thick layer of dense Kinel' clay. The excavation for the cofferdam, reaching a depth of 4 to 15 meters, was drained by deep wellpoints.

The greatest settlement – 190 mm – was observed in section No. 1 of the dam. The average settlement of all dam sections equaled 150 mm. The increase in the settlement occurring during construction corresponded approximately to the increase in the load as construction proceeded.

When the excavation trench was flooded, the onset of vertical component of water (uplift) pressure caused the section to rise by 12 mm. After the completion of the structure, when the loads had become stabilized, settlement recommenced, but at a slower rate. Tentative computations indicate that the settlement will continue for 3 to 4 years, and that over this period it will increase by an average of 50 mm. The report gives data of the inclination and horizontal displacements. These data will be analyzed and interpreted later.

SETTLEMENT OF LOCKS NOS 21-22 AND NOS 23-24 OF THE VOLGA HEP IMENI V.I. LENIN, ACCORDING TO DATA OBTAINED IN FIELD OBSERVATIONS UP TO 1 JANUARY 1958

Head of Research: S. N. Knyazev, Engineer

Research Team: A. K. Efimov, Engineer
O. A. Berestetskii, Engineer

The report gives a short description of the structures and of the engineering-geological conditions of the foundation, explains the methods applied in observing the settlements, and evaluates the accuracy of the observation data. Provisional conclusions are given, based on the preliminary analysis of the data compiled from the beginning of construction up to 1 January 1958.

The foundation of the locks consists of alluvial mixed-grained sands overlying dense Kinel' clays. The 450 × 160 m construction pit which was 450 × 160 m in area and reached a depth of 20 m, was drained by deep wellpoints. The settlements were measured with reference to a grid of fixed bench marks, and at the same time note was taken of the changes in the load acting on the foundation. The observation error can be provisionally taken as ± 20 mm. For the period from the beginning of 1953 to 1 January 1958 the maximum settlement observed in the lower locks amounted to 190 mm. It is provisionally estimated that the further settlement will amount to another 30 mm.

The settlement at the upper locks reached 180 mm on 1 January 1958 and it is estimated that further settlement will amount to another 50 – 60 mm.

DESIGN OF THE EQUIPMENT FOR OBSERVING THE DEFORMATIONS OF THE FOUNDATIONS OF THE STALINGRAD HEP STRUCTURES

Head of Research: S. N. Knyazev, Engineer

Research Team: V. A. Ioselevich, Engineer
O. A. Berestetskii, Engineer
V. N. Shaposhnikov, Engineer

The control and measuring apparatus is intended for general check observations and investigation of settlements, inclinations and horizontal displacements of the Stalingrad HEP structures, both during construction and after their completion. The equipment includes:

1) devices for observing the horizontal displacements of parts of the HEP buildings and of the spillway dam;

2) devices for observing the settlement of the run-of-river earth dam;

3) devices for observing the inclination of the powerhouse.

The devices and the method of their use (by embedding in the structure) were based on the control devices and methods used at the Volga HEP im. V. I. Lenin, with such modifications as were made necessary by the special characteristics of the Stalingrad HEP structure. The devices may be used

directly for field investigations or as reference models for the design of analogous devices for field investigation on other hydro developments.

The measuring devices were designed so as to permit their preparation in the workshop of the construction site, and also to simplify as far as possible the installation of the devices and their use in making the observations.

The equipment was prepared and installed at the Stalingrad HEP in 1958.

REVIEW OF DATA RELATING TO THE PRESSURE OF THE BACKFILL ON RETAINING WALLS SLOPING TOWARD THE FILL

Head of Research: S. N. Knyazev, Engineer

Research by: N. A. Krasil'nikov, Engineer

Retaining walls of modern hydro structures may reach heights of 40 to 70 m.

In 1951-1955 Gidroproekt carried out investigations at the Tsimlyanskii HEP with the object of devising more exact computation methods for high retaining walls built on deformable subsoils. Field investigations were made to study the behavior of such retaining walls erected on alluvial deposits consisting alternately of clay, sandy loam, and sand layers. The investigations were carried out at the upstream section No. 5 whose height was 40.5 m, and at the downstream section No. 7 whose height was 26.1 m.

The following determinations were made:

1) value and distribution of the backfill pressure on the retaining wall;

2) value and the distribution of the pressure inside the backfill;

3) pressure distribution on the subsoil beneath the toe of the walls;

4) value and nature of the deflection and of the inclination of the vertical part of the wall.

The measurement of the backfill pressure was done by wire-type soil dynamometers; deflection and inclination of the vertical parts of the walls were measured by a specially designed "coordinatometer", a plumb bob, and fastening plates.

The report presents: the readings of the soil dynamometers for the years 1951-1953 in the form of graphs; measurements obtained with the aid of the "coordinatometer" for the years 1952-1955; data on the geology of the foundation layers; characteristics of the backfill; particulars of the progress of backfill and concrete work; and data on the water level.

The data have been set out in systematic form.

CONSTRUCTION OF ROCK-FILL DAMS UNDER SEVERE WINTER CONDITIONS

Head of Research: I. N. Shcherbina, Candidate of Technical Sciences

Research Team: I. N. Shcherbina, Candidate of Technical Sciences
N. A. Pankratov, Engineer
E. M. Shvei, Engineer
N. V. Zamorina, Engineer

This report – a summary of a survey of several existing rock-fill dams – deals with various problems encountered in the investigation and observation of rock-fill dams with an impervious cone, as well as problems relating to their construction.

The two alternative methods used in modern engineering practice for the construction of rock-fill dams are: 1) the American method – in which the rock is dumped from a considerable height and then compacted by the use of a strong jet of water; 2) the Swedish method – in which the rock is placed in layers, and then compacted by means of rollers and vibration compactors and simultaneously watered. The results are given of several investigations into different methods of rock-fill placing, and the probable values are given for the settlement of differently constructed rock-fill dams. Particulars are also given of the physicomechanical characteristics of the fill and of its state resulting from the content of small-size soil particles.

The best materials for constructing the impervious core are artificial soil mixtures with a size grading determined in a similar way to that used in modern laboratories for concrete aggregate, i.e. so that the fraction consisting of particles of any size should be sufficient just to fill the voids between particles of the next (layer) size. Examples are given of soil facings (diaphragms) and cores and of the characteristics of the materials used.

The slopes of the rock-fill dams are usually made with an inclination nearly equal to the angle of repose of the material. The report deals with problems of selecting this angle, and methods for computing the stability of the dam slopes and also gives data on their seismic stability. Results are also given of observations of the deformations of the slopes and core of several constructed dams.

It is the practice abroad to add calcium chloride when carrying out earthwork under winter conditions to prevent freezing of the clayey soil.

In the U.S.S.R. a study has been made of the use of chemical additives to lower the soil freezing point, and of the influence of the additives on the soil's physicomechanical properties; some results of this study are given together with a description of methods used in the permafrost regions of the northern parts of the U.S.S.R.

The report lists questions requiring further study and investigations.

PROVISIONAL DIRECTIVES RELATING TO THE DESIGN AND THE PRODUCTION OF PREFABRICATED DRAINS MADE OF POROUS-CONCRETE BLOCKS

Head: I. N. Shcherbina, Candidate of Technical Sciences

Research Team: I. N. Shcherbina, Candidate of Technical Sciences
A. P. Voshchinin, Engineer
G. N. Shubert, Engineer

The provisional directives and explanatory notes include recommendations and remarks regarding the construction of drains, size grading of the materials, the fabrication technology, the organization of the work of placing the drains, and the basic data for planning a shop or a factory for the manufacture of porous-concrete blocks.

The directives apply to the use of porous-concrete blocks in drains of hydro structures where the drains are contiguous to the soil.

The porous blocks are made of special materials whose constituent particles are enveloped in a cement paste (mortar matrix), and adhere to each other at the points of contact, the porous mass thus formed constituting a single-layer filter.

Comparative calculations have shown that the use of prefabricated drains in earth dams cuts down costs by two thirds and greatly reduces the completion time.

Up to now porous blocks have been used only in the construction of relief wells, but by the present directives their use will be considerably extended.

These directives have been approved by the Scientific Council of the Gidroproekt and their use recommended for the design divisions of this Institute.

TECHNICAL SPECIFICATIONS FOR THE HYDRAULIC FILLING OF RUN-OF-RIVER DAM No. 40 OF THE STALINGRAD HEP

Head of Research: A. I. Ogurtsov, Engineer

Research Team: M. K. Sverchkova, Engineer
I. V. Dudler, Engineer

Technical specifications (TUiT) have been drawn up which take account of the constructional experience gained in the erection of very large earth-fill structures, and the analysis of the data obtained from experimental hydraulic-filling at the Stalingrad HEP and from the field and laboratory investigations of the Gidroproekt scientific research division.

The check figures for the compaction of the sand placing above the water level take into account the self-compaction of the soil with time.

TUiT recommend that when planning underwater hydraulic-filling, consideration should be given to the distribution of the soil according to size grade and density, depending on the distance between the dam prisms, and

the distance between the water level and the point where the fill material is discharged.

TUiT also contain directives relating to the working of the open pit, the checking of the foundations, the quality control of the work, the dimensions of the hydraulic-fill pool, the concentration of the slurry, etc.

TNISGEI IMENI A. V. VINTER ENGINEERING GEOLOGY LABORATORY

Head: L. N. Lomize, Candidate of Technical Sciences, Senior Research Worker

THE USE OF COARSE-GRAINED DETRITAL SOILS IN THE CONSTRUCTION OF HIGH EARTH DAMS

Responsible for Research: L. A. Avakyan, Candidate of Technical Sciences, Senior Research Worker

In dissected relief it is rational to build large reservoirs with correspondingly high dams. The detrital soils of the local quarries have a distinctive granulometric composition, characterized by the presence of boulders in addition to fine earth. These granulometric features determine the properties of these soils as construction material, but as these properties have not yet been studied, the efficient design of earth dams built of this material has not so far been possible.

At present a study is being made of the most typical and common detrital soils found in the local quarries of Georgia in the vicinity of the projected earth dams of the Dariali, Namakhvani, Vartsikhe, Inguri, and other hydro power plants.

This study has yielded much material regarding the properties of these soils as construction material. The soils studied belonging to many genetic varieties differ from each other in the shape and the size of their components, as well in the composition of the filler. These genetic varieties include: the debris cone and alluvial formations of the region in which the Dariali HEP dam is being erected; the alluvial shingle with clay-type filler used in the construction of the dam of the Sion reservoir; diluvial detrital soils used in the dam of the Namakhvani HEP; and alluvial formations and fine-grained soils used in the dam of the Akstafa HEP.

Investigations are being carried out at present of soils for the 240 m high dam of the Inguri HEP, based also on the study of local colluvial detrital soils.

The investigation of the properties of these soils as construction material is being made by a new method which takes into account the large size of the soil particles and uses an apparatus of large dimensions with devices for measuring permeability, and resistance to piping and shear.

This study established the following:

1. Coarse-grained detrital soils are entirely suitable for earth-dam construction; they have a number of advantages over fine-grained soils, and can be used also in the impervious elements of high structures, such as high dams.

2. The good structural qualities of these soils, and their greater friction angle, make it possible to design structures with more slender cross sections.

3. The low compressibility of these soils should reduce considerably deformations of structures caused by differential settlement, as well as the rise of pore pressure.

4. Detrital soil mixtures of fine-grained and coarse-grained components in a certain proportion have adequate imperviousness and show considerable resistance to piping.

5. Soils differing in their size grades in the proportions of coarse aggregate to the finer fraction all compact quickly and easily if the maximum particle size is 100–200 mm.

The investigation also established the following properties of detrital soils, which depend on the composition and size of the various fractions, and on the composition of the filler.

1. When the amount of the fine-earth (fraction ⩽ 2 mm) is sufficient, not only to fill the voids, but also to separate the larger particles from each other at their points of contact, the basic soil characteristics are found to correspond to those of the fine earth; thus, where there is a marked silty-clayey filler there is decreased shear resistance, accompanied by an increase in the settlement modulus with a correspondingly lower seepage coefficient.

2. If fractions below 2 mm do not entirely fill the voids and do not prevent the contact between the larger particles, the physical properties of the soil are governed by the fractions ⩾ 2 mm, and the shape of the particles is then more important than their size. Thus, angular and flat-shaped particles tend to adhere to each other, causing the soil to have a greater friction angle and lower compressibility. Rounded particles, on the other hand, tend to slide and this lowers the shear resistance and raises the compressibility as well the seepage coefficient.

Further investigations of different genetic varieties of detrital soils make it possible to classify these soils according to their use in dam construction; the use of this classification in the design of large earth dams may effect a considerable saving in construction costs and time.

STUDIES OF MORPHOLOGICAL CHANGES IN THE BANKS OF MOUNTAIN RESERVOIRS IN GEORGIA

Responsible for Research: E. E. Minervina, Candidate of Technical Sciences, Senior Research Worker

Reservoirs in mountainous regions of the U. S. S. R. are characterized by special features distinguishing them from reservoirs in the plains; the bank changes occur through a complicated interaction between various factors. The object of this investigation was thus to evaluate the extent of the expected changes in the banks, which would not conflict with the requirements

for hydro construction in mountainous regions. The work was based both on theoretical studies and on field observations.

The basic feature of mountain reservoirs is the irregular morphology of their reservoir beds, which causes a marked diversification of the engineering-geological and topographical features of these reservoirs. As a result, mountain reservoirs are of many types differing in features such as the structure of their banks, water-table parameters, water depths and net (available) head for power. Mountain reservoirs can accordingly be divided into the following types: 1) mountain-valley (deep or shallow), 2) basin, 3) canyon, and 4) mixed.

There are two principal kinds of water action on the banks according to the morphology of the reservoir, namely: wave action where there is a wide water table; and hydrostatic fluctuations where there are deeply submerged reservoir slopes. These two forms of water action affect the main morphological processes changing the banks, i.e. abrasion and landslides, the scale and character of which depend on the geological structure of the bank slopes.

The water-level regime in mountain reservoirs fluctuates widely, the daily rate of rise or fall of the water level being 5-30 cm and the annual range of the fluctuations reaching tens of meters, with a resulting adverse effect on the stability of the slopes.

A b r a s i o n is a major factor in the reservoirs of mountain-valley and basin types. Despite the relatively small surface area of these reservoirs, the waves occurring in them may reach heights of 1.5 – 2 m; these seem to be steep and of high frequency, and consequently have a strong scouring effect. The waves destroy not only clayey but also rocky banks. Observations at the Khrami reservoir showed that the rate of encroachment on the shore line is 2 – 2.5 m per year for clay loams, and 0.5 – 0.6 m per year for dolerites. The corresponding figures for the scour in depth are 0.7 – 1.0 and 0.3 – 0.4 m per year.

Another feature of morphological changes in mountain-reservoir banks is the abrasion of rocky banks.

The alternating rise and fall of the water level affects a strip of the reservoir bank extending vertically for some tens of meters, and as this prevents the formation of a shelf, any such deposits are subjected to abrasive action as the water recedes. It is therefore difficult to use the existing methods of forecasting the abrasion of the bank.

In the case of emptying the reservoir for power generation after the formation of ice in the reservoir, the vertical dynamic action of the ice cover has a destructive action on banks, particularly on banks composed of rock.

L a n d s l i d e s occur mainly in deep reservoirs (types 1 and 3), owing to the depth – up to 100 m and more – of the submerged slopes and the wide range of the water-level lowering (by emptying for power generation) (50-70 m), which results in changes in the hydrostatic equilibrium and the development of a number of physicogeological phenomena.

Under these conditions, if the reservoir bed has a steep longitudinal slope, at low water level the length of the fading zone of backwater may be as much as, say, one half of the entire length of the reservoir. This results in periodical changes in the characteristics of the upper part of the reservoir from those of a deep body of water to those of a mountain stream with erosive velocities.

The water-level fluctuations of several tens of meters and the resulting periodic immersions and changes in weight of the rock slopes cause: intensification of the weathering process; weakening of the clay layers; increased effect of hydrodynamic forces; and increased mechanical and chemical piping. The changes in the balance between the forces acting on the environment and the forces resisting such action may cause large landslides extending to a considerable height up the banks, a contributory factor being the steepness of the slopes and the general low stability of the fractured rock constituting the banks.

If large landslides occur in narrow canyons, there is a danger of additional backwater pressure for the structures situated above the normal pondage level, and in particular for the upper dams of a cascade system.

In shallow mountain-valley reservoirs there is considerable silting in the headwater by detrital material which must be flushed away for the proper upkeep of the reservoir. During flushing the swift flow of the water may scour the reservoir banks, or, the hydrodynamic forces produced by the rapid changes of the water level may cause landslides.

The question of erosion of reservoir banks is closely related to the question of conservation of level ground of the riverside which is valuable and suitable for exploitation. Mountain reservoirs often require special additional measures for bank protection, such as the anti-seepage measures, which have to be considered in the design.

The information obtained by this study was used in: 1) the design of the canyon reservoir of the Namakhvani HEP and in the assessment of the risk of massive landslides blocking the narrow river bed and thus endangering the upper stage of the system – the Ladzhanura HEP; 2) the working out of anti-seepage measures for the protection of the banks of the Khrami HEP basin reservoir, where the high seepage losses interfered with the planned river-regulation regime.

In the first case a simplified method was proposed for forecasting the bank changes caused by landslides, using the "method of the most probable angle of slope". Improvements were introduced in the landslide computations to adapt them to the design of deep storage reservoirs.

Field observations of the morphological changes in reservoir banks are continuing, with the object of working out reliable forecasting methods applicable to hydro construction in mountain areas.

TNISGEI IMENI A. V. VINTER TUNNEL LABORATORY

Head: N. L. Burdzgla, Candidate of Technical Sciences, Senior Research Worker

DIRECTIVES FOR THE DETERMINATION OF ROCK PRESSURE IN TUNNELS

Responsible for Research: G. P. Zavriev, Senior Research Worker, Candidate of Technical Sciences

Research Team: A. V. Gudzhabidze, Senior Engineer
U. Sh. Razmadze, Senior Engineer

In order to avoid having to use excessive factors of safety in the construction of tunnels for hydro structures, it is important to determine with sufficient accuracy the rock pressure and the deformation characteristics

(reaction coefficient) of the rock material. These quantities constitute basic parameters which determine the dimensions of the tunnel lining.

In most cases these parameters must be determined experimentally, and and often under field conditions. In connection with the growth of tunnel construction, TNISGEI is preparing "Directives for the Determination of Rock Pressure and Reaction Coefficient of Rock Material"*.

In view of the complicated nature of this problem and its novelty, it was necessary to verify, by a series of experiments, the correctness of the investigation method and to check the working of the measuring apparatus.

The investigation carried out by TNISGEI served as a basis for the drawing up in 1958 of the "Directives for the Determination of Rock Pressure in Tunnels"**. Experiments at the site of the Dariali HEP showed that the new apparatus designed by TNISGEI can develop a pressure of up to 30 kg/cm^2 on the rock, whereas in previous experiments it had not been possible to exceed a pressure of 5 kg/cm^2.

These directives can be divided into two parts:

1) the determination of rock pressure in fractured rock massifs;

2) the determination of the pressure in swelling ground.

For the determination of (1) it is recommended to use the indicating dynamometer devised by G. P. Zavriev. This apparatus has been successfully used by TNISGEI on the construction of a series of tunnels at the Samgori, Gyumush, and Mingechaur hydro power plants.

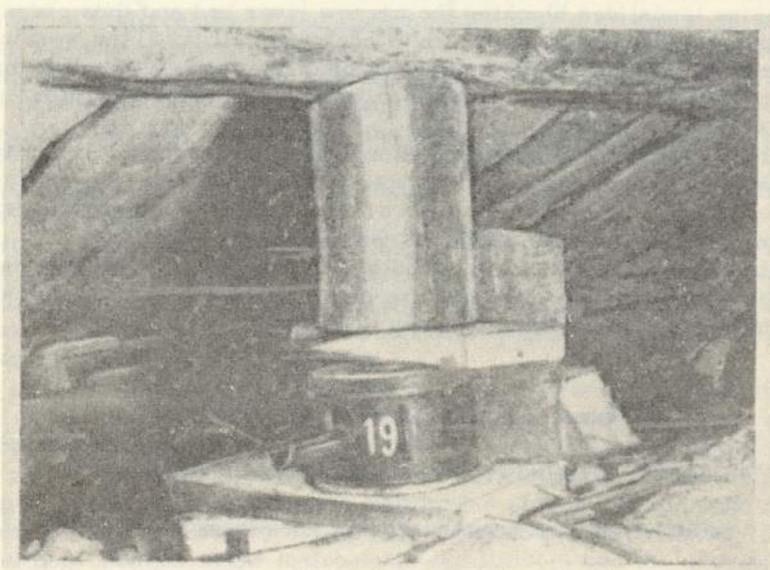


FIGURE 59. Installation of dynamometers on the timber supports of the tunnel widened to its full profile

* [Rukovodyashchie ukazaniya po opredeleniyu gornogo davleniya i koeffitsienta otpora porody.]

** [Rukovodyashchie ukazaniya po opredeleniyu gornogo davleniya v tunnelyakh.]

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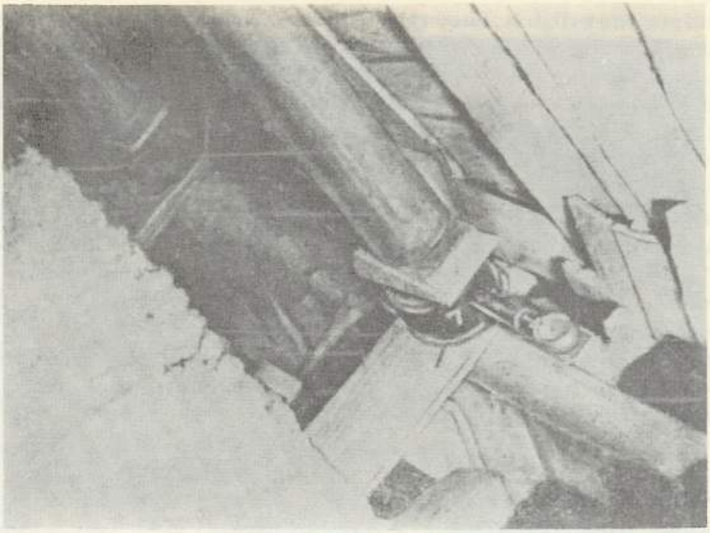


FIGURE 60. Installation of dynamometers below the timber supports of the tunnel

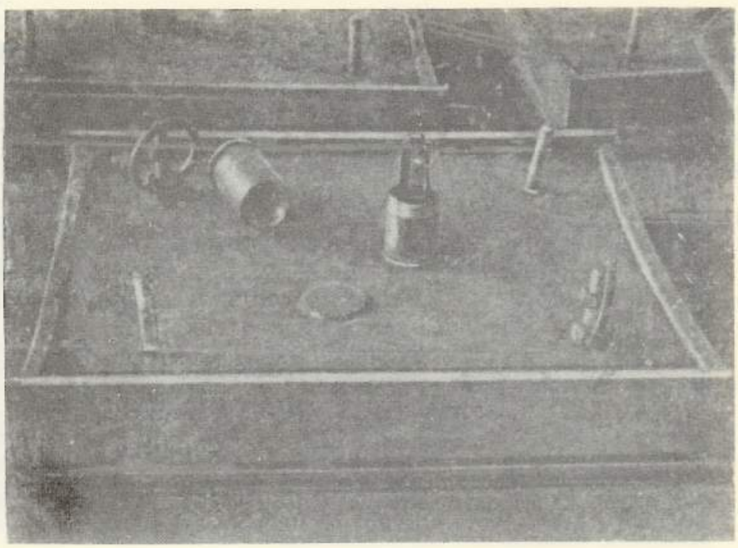


FIGURE 61. Measuring unit for determining the rock pressure

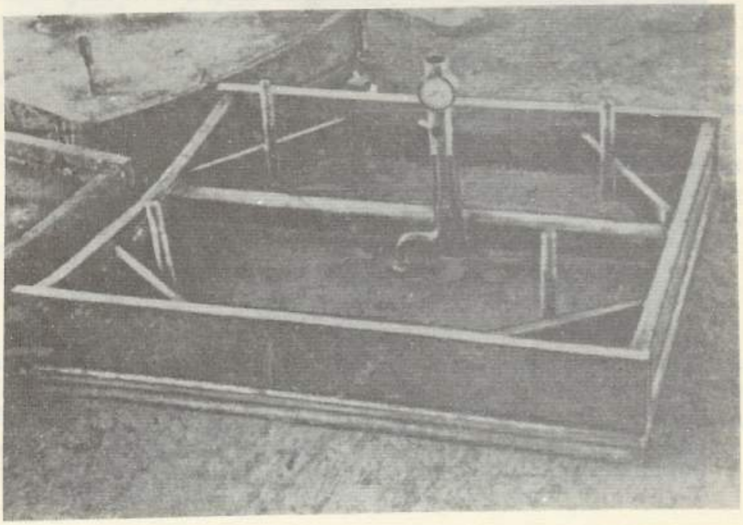


FIGURE 62. Instrument placed on the measuring unit before welding the upper cover

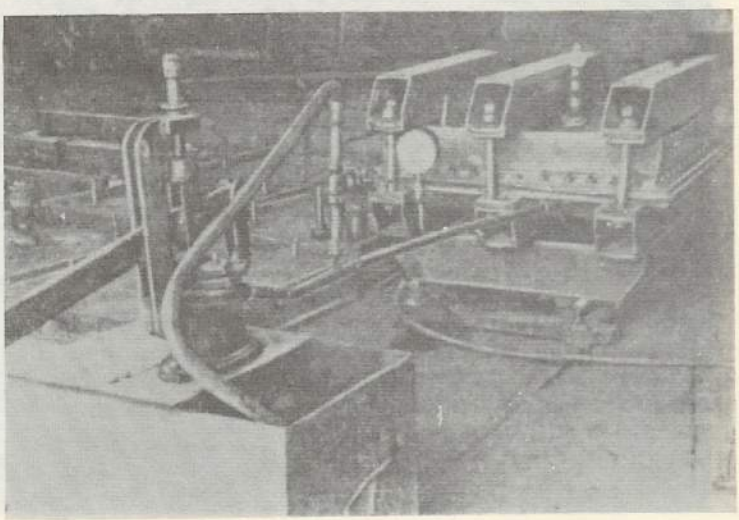


FIGURE 63. The TNISGEI calibrating apparatus

Figures 59 and 60 show the installation of dynamometers in the tunnel. In Figure 59 the dynamometer is carried by a short timber prop bearing against a longitudinal beam. The dynamometer rests on the outer surface of the concrete tunnel vault. Figure 60 shows the installation of dynamometers below the cross brace of the timber supports; for greater convenience a special cross-brace dynamometer has been devised by TNISGEI.

To measure the pressure in swelling ground, a different method must be employed, owing to the high pressures encountered which make it necessary to use the shield method for driving the tunnel. A special measuring device was constructed for this purpose by TNISGEI (Figure 61). The dimensions of the membrane adjacent to the rock face were 1.0×1.0 m, and its thickness 25 mm. Two such devices attached to the rings of the tunnel liners completely covered the external surface of the ring. The membrane deflection is transmitted by a metal rod (Figure 61, center) to a portable instrument which measures the deformation. Figure 62 shows this instrument placed on the device to which the upper cover has not yet been welded. The whole unit is stabilized by being centered on four steel spheres attached to an angle iron welded to a cross-piece by which the opposite sides of the device are connected. This angle iron constitutes a *fixed point* in relation to which the membrane deflections are measured.

The TNISGEI apparatus is of simple construction, and can be built in the workshops of the construction site.

One of the important results of these series of experiments was the equipment of a special laboratory at TNISGEI for the long-term calibration over several months of the apparatus for rock-pressure measurement.

The calibrating apparatus built by TNISGEI (Figure 63) permits the loading of a 1.0×1.0 m measuring device with a pressure of up to $70\,t/m^2$, the pressure drop over several days not exceeding 1 – 2%. During the calibration of the apparatus, carried out by TNISGEI by maintaining a pressure of 50 and $60\,t/m^2$ for four months, the readings were steady and the apparatus proved reliable.

TNISGEI determined experimentally the ground pressure in swelling ground at the Ladzhanura HEP construction. The results were extensively used by the commission headed by Academician A. A. Belyakov, dealing with the problem of the strengthening of the tunnel lining at the above construction. The conclusion reached by the commission to reinforce the existing liners instead of building a new concrete liner resulted in a saving of 30 – 40 million rubles.

The new directives permit a substantial lowering of the excessive safety factors often used in hydro-engineering tunneling practice.

TNISGEI IMENI A. V. VINTER DEPARTMENT OF SOILS AND FOUNDATIONS

Head: S. V. Dan, Candidate of Technical Sciences, Senior Research Worker

STUDY OF THE LATENT SUBSIDENCE TENDENCY OF SALINE SOILS IN THE FOUNDATIONS OF HYDRO STRUCTURES WITH REFERENCE TO THE ESTABLISHMENT OF STANDARDS FOR THEIR INVESTIGATION

Responsible for Research: S. V. Dan, Candidate of Technical Sciences, Senior Research Worker
L. N. Lomize, Candidate of Technical Sciences, Senior Research Worker

Research Team: N.A. Klapatovskaya, Junior Research Worker
V.M. Strazhevskii, Research Worker
L.M. Machaidze, Geological Engineer

The penetration of water from canals, reservoirs and other hydro structures situated on a saline soil, into the subsoil of the structures, is inevitably accompanied by the leaching out of the salts. This brings about a change in the state of consolidation and the appearance of supplementary settlements.

We call this phenomenon "latent subsidence tendency". It may sometimes be so considerable as to cause the deformation of the structure and to interfere with its normal operation, which occurred for instance at the Farkhad HEP and was investigated by VNIIG.

The study of the latent subsidence tendency was carried out by TNISGEI over a number of years in various saline soils of Transcaucasia. The soils were leached with distilled water in a pressure-seepage apparatus.

The object of the experiments was to establish the relation between the latent subsidence tendency and the size grading of the soil, the nature and the extent of the mineralization, the structure, and the acting loads. The loads applied were 0.1, 0.3, 0.5 and 1.0 kg/cm^2 and in some instances also 3.0 and 5.0 kg/cm^2.

The settlements of the soil sample during the washing were recorded and the filtrates were analyzed, these results enabling the volume of the leached-out salts to be computed.

As a result of these experiments, the following basic features of latent subsidence tendency were established:

1. Heavy soils (clays and some heavy clay loams) of nonmacroporous structure whose supplementary subsidence does not exceed 2% do not constitute a danger to hydro structures. Dangerous conditions are present mainly in medium and light clay loams and sandy loams.

2. The maximum admissible quantity of easily soluble salts in ordinary cohesive soils at moderate loads is 8%. In macroporous soils this figure must be reduced to 0.5% and the settlements may exceed 5%.

3. The gypsum content of macroporous soils must not exceed 1%, while in soils of nonmacroporous structure the maximum gypsum content may be fixed at about 10%. In this case the settlements caused by leaching depend on the gypsum content and may exceed 15%.

4. Both in macroporous soils and in soils rich in gypsum the course of the subsidence is that of a stepped downward curve, indicating "breaks" in the uniformity of the subsidence (Figure 64).

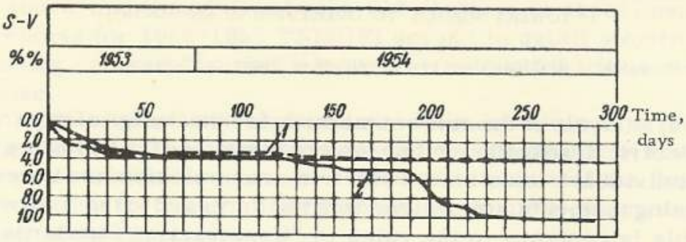


FIGURE 64. Curve of soil subsidence caused by leaching out of salts under a load of $P=0.3\,\text{kg/cm}^2$

1–leaching out of salts; 2–settlement; S–relative settlement; V–volume of leached-out salts.

5. The additional settlement of soils under compression, resulting from the leaching out of their salts, decreases with the increase in load within the range from 1 to 3 to $5\,\text{kg/cm}^2$, the settlement from 1 to $3\,\text{kg/cm}^2$ being considerably higher than that in the 3 to $5\,\text{kg/cm}^2$ range.

6. In view of the above characteristic, the settlement due to the leaching out of salts from loesslike subsoils of hydro structures endangers the foundations at loads $\leqslant 3\,\text{kg/cm}^2$.

These considerations show the possible danger to which hydro structures built on saline ground are exposed, particularly if the loads transmitted by the structure to the foundation are small.

The latent subsidence tendency of various soils needs to be thoroughly studied and further field experiments should be made, but in the meantime standards for the investigation of saline soils should be drawn up as a matter of urgency to prevent deformation in new structures.

TNISGEI IMENI A. V. VINTER LABORATORY OF HYDRO STRUCTURES

Head: Professor Yu. Ya. Shtaerman, Doctor of Technical Sciences

DESIGN OF SEPARATE FOUNDATIONS FOR ELECTRIC TRANSMISSION-LINE TOWERS SUBJECT TO OVERTURNING MOMENTS

Responsible for Research: I. I. Gudushauri, Candidate of Technical Sciences

A critical analysis of the current methods for the design of separate foundations for electric transmission-line towers* revealed the following basic defects:

a) All individual foundations of electric transmission-line towers subjected to overturning moments are designed only with regard to the limit equilibrium, although this is contrary to the rules in "Construction Standards and Regulations" (SNiP). Such a computation method is applicable only for short foundations, whose depth (h) is small compared to their width in the direction of the forces (b) acting upon them, i. e. when $h:b< 3$, as the foundations then lose their bearing capacity and overturn at small inclinations. With deep foundations ($h:b> 3$), experience has shown that overturning occurs only when the inclination has reached considerable values beyond those allowable in practical operation. In accordance with SNiP rules such foundations should be designed for limit deformation, and not for limit equilibrium conditions as at present.

b) In current practice a computation method for short foundations, based on the limit equilibrium of the subsoil and rightly used initially, is mistakenly replaced for deriving the final formulas by a method based on the *ultimate soil strength at one point*, this value being taken, without any basis for so doing, from the Coulomb equation for the reaction of the soil.

c) The current computation methods do not take into account the influence of the friction forces developing at the front and back contact faces of the foundation, and on the reaction of the soil. A similar simplification leads to the incorrect conclusion that the extrusion of the soil prism during overturning occurs both at the back and at the front faces. This conclusion is not confirmed in practice. In fact, if account is taken of the above-mentioned friction forces, the ultimate soil resistance at the front face is many times greater than at the back face. At the moment of overturning, therefore, the soil prism is forced out only at the back of the foundation. This proposition which we proved theoretically was confirmed experimentally many times by NIIOSP ASiA SSSR and by TsNIIS Mintranstroi SSSR.

d) The current computation methods are based on the physicomechanical soil characteristics and do not take into account the cohesion between the soil particles which has a marked influence on the strength of the foundation.

e) In addition to the above-mentioned primary shortcomings of the current method there are also secondary shortcomings such as the ultimate overturning load being given as an implicit function which is very inconvenient in practice, and many other secondary shortcomings which complicate the computations.

* Elektrotekhnicheskie pravila i normy (Electrical Engineering-regulations and Standards), 1933. Pravila ustroistva elektrotekhnicheskikh ustanovok (Regulations for Carrying out Electrical-engineering Installations) 1947.

On the basis of the scientific investigation carried out over a period of three years TNISGEI worked out new methods for the design of individual foundations for electric transmission-line towers. These methods are free from the above-mentioned drawbacks and conform to experimental data. In their reports for 1956-1958 TNISGEI set out in detail a critical analysis of the existing computation methods and the derivation of the new methods they propose.

On the basis of this work instructions were drawn up with tables and examples for the directives now being issued for the design of foundations of electric transmission-line towers. The main points of the instructions were approved by the members of the special committee entrusted with the preparation of the directives.

The instructions were examined and adopted by the special All-Union conference convoked by MES at the beginning of 1958.

MIIVKh IMENI V. R. VIL'YAMS CHAIR OF HYDROGEOLOGY

Head: I. I. Trofimov, Lecturer, Candidate of Technical Sciences

PRINCIPLES OF INTERPOLATION AND EXTRAPOLATION OF SOIL PROPERTIES, AND THE CLASSIFICATION OF STUDIES ON THE ENGINEERING-GEOLOGICAL CONDITIONS OF STRUCTURES

Responsible for Research: I. I. Trofimov, Candidate of Technical Sciences, Lecturer

The present lack of a theoretical basis for the interpolation and extrapolation of soil properties makes it very difficult to establish in sufficient numbers and in correct locations the exploratory mine workings necessary for the study of the engineering-geological properties of soils. A theoretical basis of this kind is also essential for the correct planning of various types of construction work and particularly in hydro engineering, reclamation, road construction and other projects where there is no actual "building site" because they extend over a large area having varying engineering-geological characteristics. The existing norms for the spacing of the exploratory workings in relation to the geological conditions of the site and to the type of structure are established on an empirical basis, and this does not ensure that engineering-geological features of importance are taken into account in the construction design or, later, in its organization and execution.

From an analysis of the process of rock formation and of the development of its engineering-geological properties, the author concludes that the interpolation and extrapolation of soil properties should be based on the two following "basic laws of geology":

1. The properties of rocks and of their contemporary state are determined during the formation process, and remain unchanged up to the present time, provided there have been no substantial changes in the environment.

2. Each genetic type of rock or soil has inherent to it a characteristic spatial formula of occurrence and stratification evident either in the present

relief or underground, with changes of composition and properties occurring according to definite laws, with a characteristic ground-water regime, and undergoing particular geological processes in its area of occurrence.

The two above postulations, considered together with the climatic zoning, form the basis also of the methods of mapping and geological evaluation of the soil properties.

The author sets out a proposed classification of engineering-geological studies of rocks and soils and of geological processes. This classification shows how far the soil studies have gone and the degree of their present reliability, and is intended to serve as a basis for the planning of research at the various stages of structural design.

There are three basis categories of these soil studies:

Category A serves as a guiding basis in the general structure design, and (A_0) in the planning of the special permanent engineering-geological observations of very large hydro structures and of reclamation works, both during construction and in operation.

Category B serves as a guiding basis in the program of engineering-geological research and investigations required for the detailed technical design.

Category C serves as guiding basis in the preliminary design; in making the choice of location of the principal structures among several possible sites; in drawing up a study program of engineering-geological research as a basis for the next stage in the designing of a project; in planning the permanent study of special engineering-geological and hydrogeological problems.

ROCKS AND RECENT GEOLOGICAL FORMATIONS AS ENGINEERING SOILS (GENERAL ENGINEERING-GEOLOGICAL CLASSIFICATION OF SOILS)

Responsible for Research: I.I. Trofimov, Candidate of Technical Sciences, Senior Research Worker

The absence of a generally accepted engineering-geological classification of rocks and of recent geological formations seriously impedes the development of engineering soil science and creates difficulties in research, design, and construction. The making of such a classification is hampered by a lack of clear understanding of the nature of the task, and to some extent also by the lack of standard meanings for some of the terms now used in different senses in engineering soil-science. This paper gives a full and objective lithological definition of disputed terms with a general engineering-geological classification of rocks and recent geological formations developed from the classification of F. N. Savarenskii.

"Soils" are defined as all rocks and recent geological formations liable to be affected by man's productive activity.

The main task of an engineering-geological classification of rocks and recent geological formations is to regroup both these as one primary category, i.e. soils with subdivisions such as categories, classes, etc. according to the similarity or equality of their engineering-geological properties,

the more detailed subdivisions being made by engineering-geological group definitions such as those defining clay and other soil types. The term "engineering-geological properties" refers to the properties of rocks and recent geological formations of significance in soil evaluation, these being mainly mechanical strength, relation to water, and resistance to weathering. All properties of rocks are determined by their genesis, composition, and environmental conditions, and this generally finds expression in their name.

The term "recent geological formations" refers to sediments, silt, and to recent as well as artifical (anthropogenic) soil formations.

"Sediments" occur both on dry land and on the land underlying a body of water and are recent deposits or formations of varying composition and genetic types which have not yet undergone all stages of lithification; since they are still in the process of formation, they are, therefore, whether on land or underwater, loose, unstable, or mobile. Examples of land formations of this type are loess and loesslike formations preserving the property of subsidence tendency and also deposits forming debris cones, taluses, shifting and loose eolian sands.

"Silts" are recent underwater deposits (marine or continental), either in the process of formation or undergoing lithification. Silts and the corresponding sedimentary rocks can be divided into genetic groups such as disintegrated and chemical; into types such as marine, lagoonal, lacustrine, and riverine; according to composition such as their chemical constituents, size grades and nature of organic residue; according to appearance; and according to variety.

"Topsoil" is a recent and still developing organic-mineral formation, mutually interrelated with all elements of the landscape. From the point of view of engineering soil-science and of lithology, topsoil constitutes a distinctive apparatus of diagenesis, through which all continental terrestrial sediments pass; elsewhere it causes the alteration of already formed surface rocks. In certain zones and under suitable conditions, topsoil formation is related to important geological processes such as loess formation, gleying, and laterite formation. For the purposes of engineering soil-science and lithology, the classification of topsoils must be based primarily on that of the accumulative and alluvial soil classes together with their differentiation into zonal types.

A more definite position in the system sediments – rocks is occupied by the artificial (anthropogenic) formations which can be differentiated into an agricultural layer and various other formations arising from man's productive activities (technogenic); artificial formations include both consolidated and as yet unconsolidated varieties.

The proposed general engineering-geological classification of rocks and recent geological formations includes all known types.

In this classification, rocks are characterized by cleavage on being subjected to shear. Semirocks are characterized by their capacity, on application of a load, to undergo an initial plastic deformation, or a bending or tensile reaction, which, under ordinary conditions, ends in cleavage. Clay and mica shale cleave in a direction perpendicular to the plane of schistosity. To the same class belong gypsum, salts, soft cohesive soil of the permafrost zone, and other soil materials which have this property of changing under pressure into a plastic state. Soils consisting of thin layers of

different composition are grouped in the class of their weakest layer. The crust eluvia of physical weathering – disintegrating rocks and semirocks – belong to the class of loose noncohesive soils.

Rocks, semirocks, soft cohesive soils and loose noncohesive soils which have the properties usual for their respective class belong to the first soil category. The second category includes rocks and recent formations that can be used as engineering soils only if special construction methods are used or after taking steps such as: the preliminary improvement of their engineering-geological properties; the improvement of the environmental conditions; and the regulation of the geological processes of their formation. Since these measures increase construction costs, this must be taken into account in the engineering-geological evaluation of this category of soils.

THE STUDY OF GEOLOGICAL PROCESSES (PART OF THE MONOGRAPH "GEOLOGICAL BASIS OF HYDRO-RECLAMATION WORKS"

Responsible for Research: I. I. Trofilmov, Candidate of Technical Sciences, Lecturer

The report deals with specific problems relating to geological processes and phenomena important in the evaluation of the engineering-geological conditions connected with the construction and operation of hydro-engineering projects.

The geological processes and the phenomena which they cause, such as surface relief, rocks, and faults, are interrelated. It is therefore incorrect to confuse the process with the phenomenon as is often done. The development of any geological process involves some type of energy – mechanical, chemical, etc. Hence the following rule can be formulated: if a geological phenomenon has a potential energy capable, under suitable conditions, of conversion into kinetic energy, it itself becomes under those conditions the cause of other phenomena, or, in other words, it becomes a process.

The report introduces the concept of "regime of geological processes". Regime here means the changes occurring during the development of the geological processes and of the phenomena they cause, under the influence of the varying intensity of the factors conditioning these processes. Geological processes can accordingly be divided into:

1) naturally developing processes: a) in an undisturbed regime, b) in a regime disturbed by human activities;

2) artificially induced processes: a) of the engineering-geological type, as caused by construction work and other engineering activities, b) of the agricultural-geological type, as caused by agricultural activities.

The geological processes of the second group do not differ essentially from those of the first group. They can be considered as natural processes of zero intensity up to the moment when the influence of human activity is felt.

The author deals with a special branch of the study of geological processes – the control of geological processes and phenomena (georeclamation), first propounded by V. M. Severgin (1765 – 1826). In georeclamation emphasis is laid on combating the geological processes causing undesirable

phenomena, and not the phenomena themselves. In principle, it is possible to control all geological processes and phenomena after the laws of their development have been studied. Two kinds of control can be distinguished: 1) direct control, i.e. the strengthening of some processes and phenomena and the weakening or complete elimination of the harmful effects of others; 2) indirect control, i.e. where it is economically or technically not feasible to change the natural conditions, adaption, by the use of appropriate structures, materials, and similar measures, to the undesirable, harmful, or destructive geological processes. The indirect method is not to be regarded as an admission of passive helplessness in the face of natural processes, but as a challenge to scientific and technical ingenuity to produce a design ensuring structural stability under unfavorable conditions.

Thus, geological processes are considered from the following aspects:

1) their role in forming and changing the surface relief (geomorphology);

2) their role in forming and changing rocks and their properties (petrography, lithology, engineering soil-science);

3) their role in engineering construction (engineering geology);

4) methods of control - georeclamation.

The report describes two types of slope deformations, which, although widespread, are little known in the engineering-geological literature.

The first type - slope subsidence - is connected with the mechanical, or chemical piping caused by ground-water action. The subsiding strip of the slope appears as one or sometimes more terraces, or as a depression along the valley, whose width corresponds approximately to the width of the depression surface of the underground stream draining the valley. The subsiding mass of the slope adheres to the main body, either along a plane inclined steeply toward the valley, or, in the case of clay soil, above the water-bearing strata, the layers are deflected along the line of adherence of the subsiding strip to the main mass.

The slopes tending to subside are those which consist either entirely or only in their lower portion of water-bearing sandy rocks - these being liable to mechanical piping, or soluble rocks (e.g. limestones, dolomites, and gypsum) being liable to chemical piping. If the impervious clayey layer above the water-bearing stratum is not exposed, slides do not often occur.

The development of slope deformations due to mechanical piping can be prevented or at least arrested by means of an anti-piping screen; this is an inverted filter which retains the particles being carried away by the water from the sandy water-bearing stratum. Slope deformation due to chemical piping can be arrested only by diverting the ground water from the subsiding portions of the slope.

The second type of deformations - collapse of dry exposed slopes of loessial, sandy, clayey or loamy soils are due to chemical piping caused by pellicular or capillary moisture. While the first type of slope deformations may appear in any climatic zone, the second type is characteristic of arid zones only.

The collapse of dry slopes is preceded by desalinization along the line where the slope joins the bottom of the valley where it is often more intensive than in the slope itself. Intensive desalinization brings about the loosening of the ground at the base of the slope, owing to the crystallization of the salts precipitated by the evaporation of soil moisture. The particles

that separate are carried away by the wind, resulting in a semioval recess being formed at the base of the slope, which finally causes the collapse of the slope. The first slopes affected are those with a southern and western exposure.

In arid zones the same process is responsible for the destruction of adobe and brick houses as well as of other structures built without a damp-course. The process is accelerated when there is brackish groundwater at such a depth that its capillary zone reaches the surface or the bottom of the valley.

The further development of chemical capillary-pellicular piping can be prevented or arrested by providing at the bottom of the slope an anti-capillary screen made of noncapillary materials such as coarse-grained sand, gruss, gravel, rubble or shingle.

IGiG AN USSR DEPARTMENT OF HYDRAULIC ENGINEERING

Head: Professor B. A. Pyshkin, Doctor of Technical Sciences, Associate Member of the Academy of Sciences of the Ukrainian S. S. R.

INVESTIGATIONS ON THE MOST RATIONAL CROSS SECTIONS OF THE OREL' RIVER DIKES AT THE DNEPRODZERZHINSK RESERVOIR

Head of Research: Professor B. A. Pyshkin, Doctor of Technical Sciences

Research Team: R. T. Slobodyan, Candidate of Technical Sciences
M. Z. Guzov, Candidate of Technical Sciences
A. Ya. Oleinik, Candidate of Technical Sciences
N. G. Pivovar, Candidate of Technical Sciences
E. S. Tsaits, Junior Lecturer
I. V. Konov, Junior Lecturer
V. A. Tkachenko, Senior Engineer
O. Ya. Garkavi, Senior Engineer
and others.

The work was carried out in close cooperation with the Kiev Institute of Hydraulic Engineering (MVO USSR).

The main aims of the project were to investigate: a) the dynamic stability of unprotected slopes of various sections of the Orel' protective dike; b) the stability of dike slopes covered by a light blanket; c) the general stability of the dike slopes; d) the seepage through various cross sections; e) the drainage arrangements and the selection of the material for the inverted filter; and f) to work out a program for observations of the dike during operation.

The following supplementary problems were also investigated: g) the morphological changes in the river banks alongside the dike alignment, and at the junction of the dike and the right bank of the Orel' River valley; h) the dam location in the most dangerous sections, in relation to the re-forming of the shore strip of the Dneprodzerzhinsk reservoir; i) the seepage through the area adjoining the dike at its tailwater side in the region of the villages of Shul'govki and Karpenki; j) the stability of the sandy slopes of the drainage

canal on the tailwater side of the dike receiving the outflow of the seepage, and the effect on their stability of fluctuations of the water level in the drainage canal.

On account of the great variety of these problems, both theoretical studies and experimental investigations were made, the experiments being carried out in wave and seepage flumes, in the EGDA simulator, and in seepage-measuring apparatus.

The following conclusions were arrived at:

1. To move the line of the right flank of the protective dike further landward on the right bank to the previously designed variation of its position in four zones by a distance equal to ten cross sections of the dike. The distance of the shift may vary from 180 to 650 m.

2. The profile of the unprotected upstream slope of the Orel' dike was so designed as to best ensure its resistance to wave action (Figure 65). The coefficient of the unprotected slope for the portion under normal headwater level is $m_2' = 45$, for the portion between the high and the maximum headwater level $m_2 = 40$, and for the portion above the maximum headwater level $m_1 = 20$.

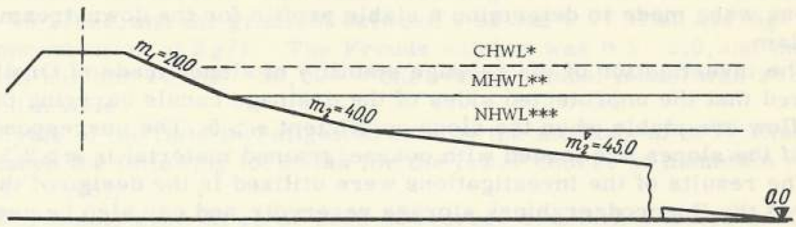


FIGURE 65. Profile of Orel' dike with unprotected upstream slope (variant I)

* [Critical headwater level]. ** [Maximum headwater level]. *** [Normal headwater level]

3. The amount of earthwork can be appreciably reduced if the steeper section of the upstream slope is covered with a light blanket of coarse-grained material. The report proposes a method for the computation of the slope stability for different gradings of the rock and rubble fill, and recommends the most suitable profiles after reviewing several alternative cross sections with unprotected and also with lightly protected slopes.

4. For the portion of the protective dike situated in the flood plain of the Orel' River, a contracted cross section with concrete slab facing of the upstream slope was selected (Figure 66). In view of the fact that in time of flood a large water area is formed in the tailwater of the dike with waves reaching 1.5 m, it is proposed that the downstream slope be covered with a riprap layer.

5. The following recommendations were made for the tailwater slope of the dike: a) for the portions of the contracted cross section situated in the flood plain, slope drainage leading to a drainage canal; b) for the portions of

broad cross section situated above the flood plain, separate canals draining the seepage water into the drainage network. A double-layer inverted filter should be used of the kind proposed in this report.

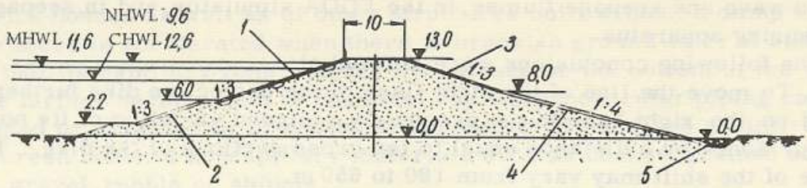


FIGURE 66. Recommended cross section of a hydraulic-fill dike in the flood plain of the Orel' River

1-reinforced-concrete slabs 10 × 10 × 0.3 m on a rubble layer, t = 0.1 m; 2-crushed-stone blanket; 3-sod laid in squares, t = 0.20 m; 4-inverted filter with stone riprap; 5-collector canal.

6. A method was worked out for computing the slope stability, and computations were made to determine a stable profile for the downstream slope of the dam.

The investigation of the seepage stability of slopes made of Orel' sands showed that the unprotected sides of the drainage canals carrying the seepage flow are stable when the slope coefficient $m \geqslant 5$. The corresponding figure if the slopes are loaded with coarse-grained material is $m \geqslant 2.5$.

The results of the investigations were utilized in the design of the Orel' dike of the Dneprodzerzhinsk storage reservoir, and can also be used in the design of other protective structures for similar conditions.

The results of the present work, when introduced into hydro-engineering practice, will undoubtedly improve structural reliability while at the same time lowering construction costs.

IEVKh LABORATORY FOR HYDRAULIC ENGINEERING STRUCTURES ACADEMY OF SCIENCES OF THE KIRGIZ S. S. R.

Head: K. F. Artamonov, Candidate of Technical Sciences

MORPHOMETRIC CORRELATIONS OF THE STABLE PORTIONS OF MOUNTAIN-RIVER CHANNELS IN KIRGIZIA

Responsible for Research: A. N. Kroshkin, Junior Research Worker

The study of mountain rivers with a view to the sound designing of hydro-engineering works is essential to further projects for the economic development of mountainous regions such as: irrigation; power production for agriculture; irrigation of meadows for hay growing and grazing; and power supply for cattle raising. The mountain rivers of Kirgizia can be divided into three groups

at a confidence level of 10 – 0.1 % according to the extent of the streamflow leading to morphological changes in the river bed: 1st group, $Q_r = 10 - 30\,m^3/sec$; 2nd group, $Q_r = 30 - 70\,m^3/sec$; 3rd group, $Q_r = 70 - 350\,m^3/sec$ and more. According to their gradient, the mountainous reach of such rivers can be divided into four parts: the headwaters ($i = 0.10 - 0.40$); the upper course ($i = 0.08 - 0.12$); the middle course ($i = 0.04 - 0.08$); the lower course ($i = 0.01 - 0.05$).

The two last-mentioned – the medium and the lower courses – are the most important for the economic exploitation outlined above.

In 1956-1958 a study was carried out of the morphometric correlations of stable alluvial river beds in Kirgizia, by laboratory investigations made on an open-platform model, and by field investigations. Measurements were made of the width and depth of the river channel in river stretches that were straight or only slightly curved, and free from large boulders or rock fragments. The average particle size of the river-bed load was determined by sieve analysis by the measurement of individual stones. The gradient was measured by leveling the surface of the stream over sections 50 – 200 m long. The model analysis was carried out according to the method devised by M. A. Velikanov and S. T. Altunin.

A modeling coefficient of 20 was taken for the bed load and for the linear dimensions of the model. The water discharge in the model varied between 10 and 40 l/sec, and the gradient between 0.03 and 0.07 at an average bed-load concentration of 3 g/l. The Froude number was 0.5 - 2.0, and the Reynolds number 14,000 – 50,000. The river-channel model was 20 m long and 0.80 m wide.

Analysis of the field-investigation data by the methods of S. T. Altunin established the following formulas for the lower course of mountain rivers ($i = 0.01 - 0.05$):

1. For stable alluvial river beds of average erodibility (slope coefficient $m > 1.0$):

$$B = A\frac{Q^{0,34}}{i^{0,17}}. \qquad (1)$$

2. For stable alluvial river beds of low erodibility (slope coefficient $m = 1.0 - 0.3$):

$$B = A\frac{Q^{0,30}}{i^{0,5}}. \qquad (2)$$

In these formulas:

Q = the extent of streamflow (m^3/sec) leading to morphological changes in the river bed (at a confidence level of 10 – 0.1 %);

i = water-surface gradient;

B = width of river at water line, in m;

A = 1.8 – 3.0, a coefficient depending on the slope inclination, the average particle size of the deposited river load which determines the roughness of the river channel, and the value of Q (higher values of A correspond to rivers of the third group, medium values to the second group, and low values to the first group).

Using the general morphometric formulas of M. A. Velikanov, as well as the results of field and model investigations of river channels of low erodibility at the middle and lower courses ($i = 0.01 - 0.08$) of the rivers belonging to the 1st and 2nd streamflow groups ($Q = 10 - 70$ m/sec), the following formulas were established:

$$\frac{B}{d} = 1.74\left(\frac{Q}{d^2\sqrt{gdi}}\right)^{0.46}, \tag{3}$$

$$\frac{H}{d} = 0.32\left(\frac{Q}{d^2\sqrt{gdi}}\right)^{0.34}, \tag{4}$$

where g = acceleration of gravity, in m/sec^2;
H = mean depth of the stream, in m;
d = mean particle size, in m, of the river-bed load, which determines the roughness of the river channel; according to field data, d can be taken as the size of the largest fraction constituting 20% of the total weight of all particles making up the river-bed load, or, for rivers of the first and second groups, d can be provisionally taken as being equal to $5.50\, i^{0.80}$, in m.

Other symbols as in formulas (1) and (2).

Modifying formula (3), we obtain an approximate equation for determining the width of stable river beds of the lower reaches of mountain streams:

$$B = 1.25\,\frac{Q^{0.40}}{i^{0.20}\,d^{0.10}}. \tag{5}$$

Formulas (1), (2) and (5) are recommended for the determination of the parameters of stable alluvial sections of mountain-stream beds.

CHAIR OF HYDRODYNAMICS OF THE BPI IMENI I. V. STALIN

Head: Prof. M. F. Makarochkin, Doctor of Technical Sciences

DESTRUCTION OF SLOPES OF SOIL-RECLAMATION CANALS UNDER THE ACTION OF HYDRODYNAMIC PRESSURE

Responsible for Research: Prof. M. F. Makarochkin, Doctor of Technical Sciences

Research by: Yu. A. Sobolevskii, Candidate of Technical Sciences, Lecturer

Investigations were carried out to develop methods for the design of slope of soil-reclamation canals operating under various geological conditions as prevailing in Belorussia.

Most intense destruction of slopes is observed in the initial period of canal operation, i.e. soon after the excavator finished its earth-cutting work and the ground-water table is sharply lowered. In this period the hydraulic gradients in the bulk of the slopes attain peak values.

The technique of field experiments was adapted to these particular conditions.

Special pipes (called here filters) are lowered by drilling methods in areas close to the slopes of the canals being constructed. Other pipes of smaller sizes are lowered (by the same method) into the slopes proper. The pipes are intended for measurements of ground-water table fluctuations. These measurements are continued until the fluctuations in the ground-water table become independent of meteoralogical factors.

Parallel with these ground-water records, periodical level surveys of the slope cross sections are carried out. By combining the results of both types of measurements, conclusions may be drawn as to the extent of deformation and destruction of the slopes under the action of hydrodynamic ground-water forces. The investigations were carried out for different underground conditions of the soil being drained.

The results of these investigations in conjunction with theoretical considerations will be used in the development of methods for calculating the resistance of canal slopes to hydrodynamic forces.

The present study deals with the problem related to sandy slopes subject to water seepage.

The field investigations on the state of slopes yielded data on the extent of slope deformations for different soil varieties. The study gives a detailed classification of cases of destruction of reclamation-canal slopes under conditions prevailing in Belorussia. The data were included in the technical specifications for the design of soil-reclamation structures developed by the Belgiprovodkhoz.

BUILDING MATERIALS

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR CONCRETE

Head: Professor V. V. Stol'nikov, Doctor of Technical Sciences

THE EFFECT OF AGE ON THE BASIC PROPERTIES OF CONCRETE USED IN HYDRO STRUCTURES

Responsible for Research: A. S. Gubar', Candidate of Technical Sciences, Senior Research Worker

This is a continuation of investigations carried out by the Laboratory for concrete of VNIIG, in connection with the new trends arising in hydro construction to use grades of concrete having a hardening age of 180 days and more.

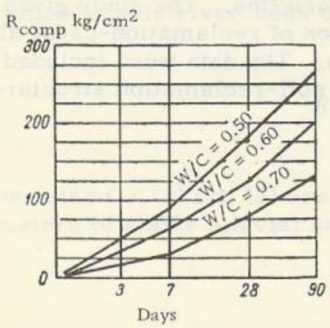


FIGURE 67. Diagram showing increase in compressive strength

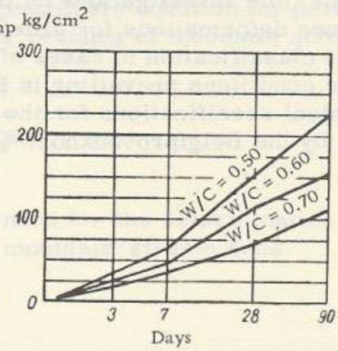


FIGURE 68. Increase in compressive strength for various mixes of concrete stored under optimum moisture and temperature conditions

The research conducted in 1958 dealt with the effect of age and variable W/C (water-cement) ratio on the basic properties (strength, watertightness and frost resistance) of concrete mixes prepared from slag-portland cement and pozzolana-portland cement. The tests were performed on concrete mixes with and without addition of surface-active agents and with addition of air-entraining and plasticizing agents. The specimens were tested for strength, watertightness and frost resistance at hardening ages of 7 days, 28 days, 3 months, 6 months and 1 year.

Figures 67, 68, 69 show data obtained so far in connection with the increase in compressive strength of slag-portland cement concrete mixes of various proportion stored under optimum temperature and moisture conditions.

It can be seen that the rate of increase in compressive strength (under given hardening conditions) is higher at the later stages of hardening (over 28 days) for slag-portland cement concrete than for concrete made of conventional portland cement. The strength of portland-cement concrete at a hardening age of 90 days increases by 15 to 40% compared with its strength after 28 days, whereas for slag-portland cements of the same hardening ages strength increases from 35 to 65%.

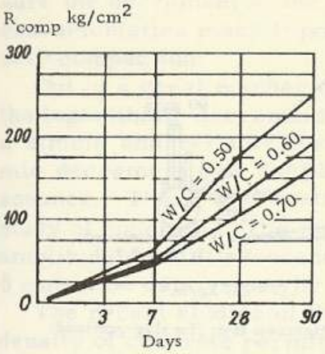


FIGURE 69. Increase in compressive strength under optimum temperature-moisture conditions

Thus, the time factor of strength increase, important as it is, is still more important for concrete made of slag-portland cement than for concrete made of portland cement. The study of effect of age on watertightness of concrete made of slag-portland and pozzolana-portland cement was carried out according to the GOST-4800 testing procedure, which makes it possible to obtain directly those indexes of watertightness which are used in practice.

Preliminary results of watertightness tests for slag-portland cement concretes are shown in Figure 70. It can be seen that in the period between 28 and 30 days the watertightness increases considerably - more than four times. It is very important to note that, with age, the rate of increase in watertightness is much higher than that of strength.

The tests results prove the possibility of obtaining a marked reduction in the cost of concrete for hydro structures, by considering the increase in watertightness with age, a fact which is of particular importance in modern hydro construction, where in most cases it is the watertightness requirements which determine quality and cost of concrete. The study of effect of age on frost resistance was carried out on a series of different concrete mixtures with various water-cement ratios, addition of surface-active agents etc. Condition of samples during the tests was controlled by observing the changes in the dynamic modulus of elasticity. Frost resistance was also evaluated directly from the decrease in strength during the tests.

The results of the study may be used both in design and practice for determining grades of concrete having a hardening age of 180 days and more.

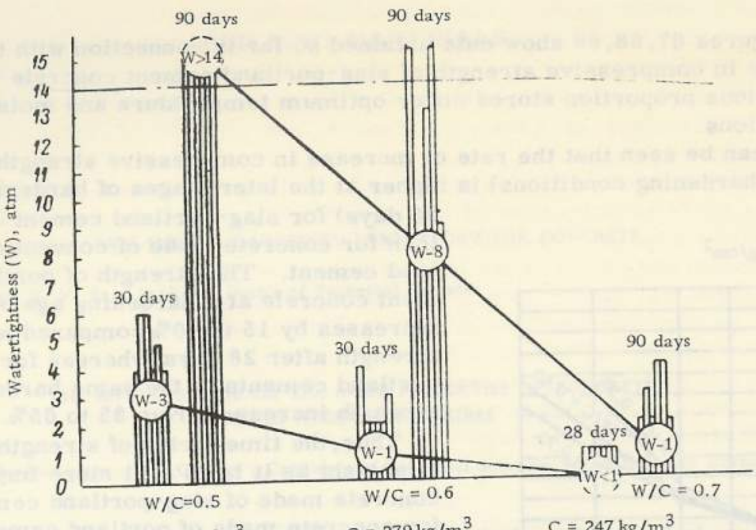


FIGURE 70. Diagram showing preliminary results of watertightness tests (for slag-portland-cement concrete)
[C = cement content.]

STUDY OF EFFECT OF SELF-COMPACTION OF CONCRETE ON ITS STABILITY UNDER VARIOUS CONDITIONS OF EXPOSURE

Responsible for Research: Professor V. V. Stol'nikov, Doctor of Technical Sciences

Research by: A. S. Gubar', Candidate of Technical Sciences, Senior Research Worker
B. V. Sudakov, Senior Engineer

Spontaneous increase in density (self-compaction) is an important factor affecting the stability of concrete at different degrees of exposure, especially its watertightness and frost resistance. The study deals with the relationship between self-compaction of concrete and its durability under influence of varying temperatures and moisture. The laboratory devised a new method for the study of self-compaction and the influence of moisture on the state of concrete. The new method makes use of the technique of plotting resonance-frequency curves of concrete specimens subjected to the action of external factors.

The concrete laboratory proved for the first time that there is a definite relationship between the moisture of the test specimens and their resonance frequency. The relationship is of a complicated nature and is connected with changes in the density of concrete as a result of penetration of water and swelling of the cement stone (matrix). This relationship permits the

resonance method to be applied to the study of self-compaction caused by swelling, etc.

The investigations have proved that by plotting resonance-frequency curves it is possible to study the increase in density of concrete upon wetting and decrease upon drying.

Thorough laboratory investigations proved the utility of this method in investigating the phenomenon of self-compaction. Moreover, it was found that, apart from measuring the resonance frequency, it is necessary to measure the decrement of the damping of oscillations in the specimens. The two characteristics make it possible to obtain a clear picture of the process of self-compaction.

Out of a great number of well-known methods for the determination of the logarithmic decrement of oscillation damping in these investigations, a simple and reliable method was used for the determination of logarithmic decrement δ of oscillations from the width of amplitude peak of resonance. There are certain difficulties in applying this method for the study of concrete. The main difficulty lies in the appearance of a double amplitude peak of resonance. For some types of these peaks the decrement δ cannot be determined by this method.

The recent studies of the relationship between moisture and structural density of concrete permit the study of self-compaction and its dependence on various factors. The laboratory studied such basic factors as water-cement ratio, cement contents, effect of various additives (surface-active agents, water-repellant and water-retaining substances), etc.

The study of these problems required numerous tests. Parallel with the study of the effect and kinetics of density by the method of frequency-resonance curves and of logarithmic decrement of oscillation damping, the laboratory also investigated the technical properties of concrete.

The purpose of the study has been to establish the physicochemical mechanism of self-compaction with a view to controlling and utilizing this phenomenon in practice.

CORROSION RESISTANCE OF CEMENTS WITH ADMIXTURE OF FUEL CINDERS

Responsible for Research: V. V. Kind, Candidate of Technical Sciences, Senior Research Worker

The possible use of finely dispersed fuel cinders as admixtures to cement or concrete is of great importance to hydro construction, as it permits considerable savings in cement. Until now, however, the influence of cinders obtained from U.S.S.R. thermal plants on the specific technical properties of cement and concrete has not been investigated. The purpose of this work was to solve this problem by studying the basic properties of cements and concretes containing power-plant waste cinders of different composition.

In 1958 the influence has been investigated of admixtures of various cinders on the corrosion resistance of portland cements place in sulfate solutions and in soft waters. For this purpose 20 mixes were prepared of a pure clinker portland cement with 25, 30 and 35% of cinders from the

Shchekinsk, Kashira, Moscow No. 11 and 12, Irkutsk and Krasnoyarsk thermal power plants, and 2500 specimens prepared from these mixes were placed in sodium and magnesium sulfate solutions and in soft running water and cured under normal moisture conditions.

Results of investigations performed in 1958 make it possible to work out instructions on use of cinder as admixtures to concretes placed in aggressive surroundings.

In addition to the experimental investigations in 1958, a theoretical research was conducted on components of cinders from various U.S.S.R. thermal power plants to permit classification of cinders as admixtures to concrete used in hydro structures.

ON THE POSSIBLE USE OF BLAST-FURNACE SLAGS AS AGGREGATES FOR HYDRAULIC CONCRETE

Responsible for Research: I. L. Znachko-Yavorskii, Candidate of Technical Sciences, Senior Research Worker

The possible use of blast-furnace slags as aggregates for plain and reinforced concrete, would be of considerable advantage to the national economy. During the years 1953-1955, in the U.S.A.; 78 to 80% of blast-furnace slags were used as aggregates*. In Germany the amount of slags used as aggregate for concrete is higher than that used for the manufacture of cement. In the U.S.S.R., on the contrary, up to 95% of granulated slags are used for the production of cement, while the production of slag aggregates for concrete amounted in 1955 to 800,000 m^3, and in 1957 to 1,270,000 m^3 only.

Old dump slags from metallurgical plants, available in quantities of over one hundred million tons and increasing daily, represent a huge potential source of raw materials for the production of slag aggregates.

From the practice of southern building agencies and from investigations by the YuZhNII, it can be seen that the cost of locally available crushed slag amounts to 20-30 rubles per 1 m^3 against 60-80 rubles per 1 m^3 and 100-120 rubles per 1 m^3 for local and imported crushed granite stone, respectively. The prime cost of finely ground slag (sand) amounts to 2-4 rubles/m^3 against 20-30 rubles/m^3 for locally available sand or for sand brought from other deposits. U.S.S.R. standards provide for the use of slag aggregates (crushed slag) or for road-making concrete. Blast-furnace slags for hydraulic concrete are used only as finely ground hydraulic and filling admixtures.

The objective of this work was to investigate the possibility of using blast-furnace slags from the Dneprodzerzhinsk works as coarse and fine aggregates for hydraulic concrete. Slag aggregates, as well as hydraulic concretes prepared with these aggregates in the laboratory, were subjected to special chemical, petrographical and physicomechanical tests.

* These figures do not include the foam-type slags (lightweight aggregates) whose amount is 7.5 to 9 % of the total quantity used.

The investigations yielded the following:

The density of conventional crushed-slag particles does not correspond to GOST 4797-56 specifications. It is smaller owing to the lower density of the vitreous and porous components. This slag, however, meets the specifications for size grading, content of impurities, easily removable particles, weak grains and frost resistance.

Standard finely ground slag, though satisfying the requirements of GOST 4797-56 for size grading and impurities, does not satisfy the requirements for the content of particles easily removable (by washing out).

Granulated slag, (studied for the sake of comparison) meets GOST 4797-56 requirements for sands, but has a high water absorption because of its porous structure.

The study of slag aggregates of concretes (with a slump of 4-6 cm and a water-cement ratio of 0.60 to 0.70 for external and of 0.80 for internal parts of structures permits the following conclusions.

Slag aggregates can be used for hydraulic concrete. Concrete with slag aggregates is characterized by a high watertightness and an increased tensile strength. Its use seems suitable for the internal parts of structures and for submerged parts requiring a high degree of watertightness.

Where high frost-resistant concrete is required, slag aggregates should not be used without surface-active admixtures, as their inhomogeneous structure and different density might cause seggregation of the concrete mix, and hence reduction of its frost resistance. They can, however, be used in concretes with less severe requirements for frost resistance.

Concretes with standard coarse and fine slag aggregates without surface-active additions require a higher cement content than concretes prepared from natural aggregates. This fact, however, does not apply to concretes with high water-cement ratios and is the result of using finely ground slag containing a large amount of easily removable particles.

Concretes of the same W/C ratio, same slumps, and made of the same crushed-slag coarse aggregate but with standard natural sand (as fine aggregate) require the same amounts of cement as concretes prepared with natural coarse and fine aggregates.

This study should be considered as the first stage of research and gives a positive answer to the problem of using ordinary blast-furnace slags from the southern regions as aggregates for hydraulic concretes.

CONCRETE MIXES FOR PRESTRESSED REINFORCED-CONCRETE LARGE-DIAMETER PIPES

Responsible for Research: M. K. Smerdov, Senior Engineer

This work deals with studies on various possible concrete mixes for reinforced-concrete pressure pipes of large and medium diameters, the basic data being obtained from the study and systematization of available literature.

For the manufacture of reinforced-concrete cores, the Soviet Trust Zakmetallurgstroi used concrete grade 350-400 with a content of 500 to 550 kg/m^3 of pozzolana cement grade 500. The aggregates for this concrete

were made of crushed stone (maximum particle size 20 mm) obtained from local rocks having a compressive strength of 800 kg/cm^2. As no pure quartz-sand deposits were available, tailings from crushed stone of a grain size of 0.1 to 5 mm were used as fine aggregates. In order to obtain a highly dense concrete the W/C ratio has to be reduced to a minimum, i.e. to 0.4-0.5, corresponding to a slump of 0.5 cm. The lower range of the W/C value is limited by the requirements for workability which can further be increased by adding SL plasticizers [see p. 160]. Practice shows that this plasticizer considerably increases the flowability of the concrete mixes with the same W/C ratio, thus increasing the density of concrete, eliminating superficial flaws, and rendering smooth the surface of pipes.

East German factories manufacture reinforced-concrete pressure pipes of a diameter of 0.5 to 2 m. The concrete used has the following characteristics and composition: grade – 450 kg/cm^2, a content of 600-650 kg/m^3 of cement, 830 kg/m^3 of gravel up to 7 mm size, 25 kg/m^3 of quartz sand up to 0.2 mm in size and 625 kg/m^3 of crushed stone; the W/C ratio is 0.4.

For the high-pressure aqueduct of San Diego in the U.S.A. a concrete mix with a slump of 6.5 cm was used for all pipes. Air-entraining agents were added, the amount of the entrained air being limited to 2%. The maximum particle size was 38 mm, and the cement content 335 kg/m^3. The pipes satisfied the requirements for high-pressure pipes. The water conduits had sufficient density and a smooth flawless surface.

Hungarian factories manfacture reinforced-concrete pressure pipes of diameters up to 2 m. Composition of concrete: maximum size of coarse aggregates 20 mm; cement content 540 ± 20 kg/m^3, water-cement ratio 0.46.

It can be seen that various countries use concrete mixes of greatly varying compositions.

It is necessary to continue research on the optimum composition of concrete mixes, considering the available technological possibilities of pipe manufacture.

DESIGN OF INSTRUMENTS AND DEVICES FOR SCIENTIFIC RESEARCH

Responsible for Research: V. B. Sudakov, Senior Engineer

After preliminary work in 1957, in 1958 a prototype model has been constructed of an instrument for field tests of concrete strength. As distinct from other instruments of the same purpose which utilize the relationship between the limit of compressive strength of concrete and the values of residual deformations caused by impacts of different devices in the shape of a punch against the surface of concrete, the new device makes use of the relationship between the strength of concrete and the amount of rebound movement of a steel hammer dropping on the concrete surface (scleroscopic-hardness measurement).

Experimental tests of this device confirm that this principle can be utilized for approximate estimates of concrete strength. This requires,

however, certain constructional changes in the design, e. g. use of a stronger spring, changes in the design of the hammer, arrangement for recording the amount of rebound, etc.

These experiments confirmed that the amount of rebound is markedly affected by the nature of concrete and its state at the time of experiment.

COMPILATION OF COMPARATIVE DATA ON THE TECHNICAL LEVEL OF THE EXISTING SOVIET NORMS AND TECHNICAL SPECIFICATIONS FOR BUILDING DESIGN AND STANDARDS FOR BUILDING MATERIALS AND PRODUCTS, AND THE ANALOGOUS NORMS AND STANDARDS IN FOREIGN COUNTRIES

Responsible for Research: I. L. Znachko-Yavorskii, Candidate of Technical Sciences, Senior Research Worker

The purpose of this work was to compare GOST 4795-53 and GOST 4797-56 specifications for hydraulic concrete with the existing corresponding foreign norms. The special Instructions of the Department of Scientific Research Works of the Academy of Building and Architecture of the U. S. S. R., as received on 12 May 1958, did not include data on this subject.

The first and basic stage of this study consisted in making available the required foreign standards and norms. It turned out that the agencies, which might possess the necessary material, such as the Library for Foreign Standards of the All-Union Scientific Research Institute of the Bureau for Standards, Measurements and Measuring Instruments in Moscow, the Central Institute of Scientific Information of the Academy for Building and Architecture of the U.S.S.R., and a number of other institutions are very short of this material. The available material has been collected and is now being analyzed. Measures have been taken to obtain the required material from abroad.

In the first stage the GOST 4795-53 and 4797-56 have been compared with available American standards.

Though it is not yet possible to suggest any changes in the Soviet standards because of limited amount of work completed, a number of valuable points can already be mentioned which might be useful in Soviet practice, such as, for instance, specifications for the use of local slag aggregates for hydraulic concrete with a resulting high economic efficiency; the use, alongside with standard specifications, of experimental specifications to be subsequently checked, so as to accelerate introduction of new standards reflecting the scientific and technical progress. Certain differences have also been noticed between Soviet and American methods and criteria referring to testing materials and layout of standards.

It is therefore necessary to continue the study of standards of other countries, if comprehensive results are to be achieved. The work done so far confirms the usefulness of such attempts.

THE INFLUENCE OF "DEKOBETON" ADMIXTURES ON THE IMPROVEMENT OF THE TECHNOLOGICAL PROPERTIES OF CONCRETE MIXES AND OF THE TECHNICAL CHARACTERISTICS OF HARDENED CONCRETE

Responsible for Research: A. S. Gubar', Candidate of Technical Sciences, Senior Research Worker

The technological characteristics of concrete mix, such as plasticity and workability, as well as technical characteristics of hardened concrete, may be improved by incorporating in the mix special additives.

This study dealt with investigations on the influence of "Dekobeton" admixtures obtained by the VNIIG from the All-Union Trust Raznoeksport.

As can be seen from the specifications, Dekobeton increases plasticity and watertightness of concrete.

In order to establish the effectiveness of the Dekobeton admixtures they have been compared with two other Soviet admixtures, namely neutralized air-entraining resin (NAR) and spent sulfite liquor (SL).

The conclusions of the research are the following:

1. NAR and SL admixtures increase plasticity and workability of concrete as much as three times, while Dekobeton increases it only 1.5 times, compared with a plain concrete prepared without any admixtures.

2. With the same plasticity (slump 3-5 cm) and proportion of concrete mix (1:6.8), the W/C ratio of mixes containing NAR and SL admixtures can be reduced by 10% compared with a (reference) concrete without admixtures. By adding Dekobeton (other conditions being the same) the W/C ratio, could be reduced only by 5%, i.e. half of that obtained when using NAR or SL admixtures.

3. Concrete containing Soviet-made NAR or SL admixtures is more watertight than concrete made with Dekobeton. Most efficient proved the air-entraining NAR admixture: the watertightness of such a concrete is three times as high as that of concrete containing Dekobeton.

4. All of the specimens were subjected to 150 cycles of freezing and thawing. The specimens containing Dekobeton admixtures lost 63% of their strength and 10% of their weight, compared with the reference specimens, i.e. they did not withstand the test.

5. Specimens containing the air-entraining NAR admixtures proved on tests to have the highest frost resistance. After 150 cycles their strength was 141% of that of the reference specimens.

INSTRUCTION ON CONCRETING IN WINTER WITHOUT HEATING BOTH THE MATERIALS AND THE CONCRETE PLACED (COLD CONCRETE)

Responsible for Research: V. B. Sudakov, Senior Engineer

This study based on the research by VNIIG and on conclusions drawn from data available in literature on concretes with chloride admixtures made it possible to clarify certain points referring to the nature of these concretes and to the fields of their possible use. These points are reported

as notes and supplements to the Temporary Instructions (I-207-55 MSPMKhP) on the use of concrete, hardening under frost conditions, with admixture of salts, and to the Temporary Instructions (I-200-56 Minstroi) on preparation and use of concrete hardening under frost conditions, with admixtures of chlorine salts.

The notes and supplements to Instructions I-207-55 and I-200-56 contain suggestions for better editing of the instructions as well as corrections and supplements to certain points, etc., which might be useful in drawing up new "Instructions on the use of concrete hardening under frost conditions, with admixture of salts".

The notes and supplements to either Instructions have been supplied to the Scientific Research Institute for Concrete and Reinforced Concrete of the Academy of Building and Architecture of the U.S.S.R. which has been charged by the Government Committee for Construction with the preparation for publication of All-Union Instructions on the use of concrete, hardening under frost conditions, with admixture of salts.

STUDY OF CERTAIN TECHNICAL CHARACTERISTICS OF CONCRETE USED FOR THE MAIN STRUCTURES OF THE BRATSK HYDRO POWER PLANT, AND TECHNICAL ASSISTANCE GIVEN

Research Team: Ts. G. Ginzburg, Candidate of Technical Sciences, Senior Research Worker
R. E. Litvinova, Candidate of Chemical Sciences, Senior Research Worker

In 1958 studies continued both at the Laboratory for Concrete of the VNIIG and on site, on the technology of preparation of concrete for the Bratsk hydro power plant. The following studies were completed during this period.

1. The existing technical specifications for cement worked out by the concrete laboratory of the VNIIG have been modified as a result of study of actual working conditions at the structures of the Bratsk HEP.

The modified technical specifications have been approved by Giprotsement and higher authorities.

2. The important problem of opaline occurrence in the sand-gravel quarries of the construction project has been investigated in situ by the Angara survey team of the research institute.

3. The construction laboratory tested the concrete developed by the VNIIG, for the internal areas of the structure. This concrete has a cement content of 180 kg/m^3.

4. Together with the construction laboratory, VNIIG investigated on the site the effect of nonuniform grain-size distribution of sand obtained from the river quarry No. 1, on the ultimate W/C ratio of the basic concrete mixes. The possible variations in the cement content per 1 m^3 of concrete as a result of the primary state of sand have also been studied. The results obtained from the study of quarry No. 1 sand mixed with quarry No. 26 sand (from the Zui Island) show that the changes in the water-cement ratio do not exceed the permissible limits for given concrete mixes. However,

for other concrete mixes, in particular for concrete with MRZ 250 additives, a change in the W/C ratio is not admissible.

5. As a result of field investigations carried out by VNIIG in collaboration with the construction laboratory and Mosgidep, and on the basis of geological reports, a deposit has been selected for supply of aggregates suitable for frost-resistant hydraulic concretes (MRZ 100) for the Bratsk HEP structures. This selection has been confirmed by the Glavgidroenergostroimontazh of the MES SSSR

6 The VNIIG in collaboration with the building laboratory investigated, on the site, the possibility of incorporating large-size crushed-stone aggregates into the concrete mix. The experiments prove that this method permits considerable savings in cement and sand, which is of particular importance for the Bratskgesstroi, where the local sand-gravel deposits are not sufficient to cover the needs of construction.

7. The influence of the time lag between the preparation of concrete and its placing, on the quality of concrete, has been the object of a particular investigation. The results of experiments do not confirm the prevailing opinion that the strength of concrete decreases considerably if the time interval is higher than 2 hours. Neither is the watertightness reduced if concrete is properly compacted.

8. The concrete laboratory of VNIIG designed a precise and reliable apparatus for determining the amount of heat release in cements and concretes. The reliability of this new device has been checked. Data obtained on heat release in cement mortars and concretes can be used for thermal calculations of concrete placing at the Bratskgesstroi.

9. Moduli of instantaneous deformation as a function of age of concrete have been determined for concretes of basic compositions for the Bratskgesstroi structures (concrete for internal, submerged, and frost-resistant structures).

10. Large cylindrical metal molds have been prepared for determining the strength of concrete.

INVESTIGATIONS OF CONCRETE USED FOR THE MAIN STRUCTURES OF THE KRASNOYARSK HEP

Responsible for Research: Ts. G. Ginzburg, Candidate of Technical Sciences, Senior Research Worker

I. The following experimental work has been performed in 1958 at the concrete laboratories, on proportioning concrete mixes for the main structures of the Krasnoyarsk HEP:

1. For the upstream face of the dam a composition of the submerged concrete has been selected, based on aggregates from the Shumikha Island deposits and on slag-portland cement from the Krasnoyarsk plant.

2. For the upstream face in the area of fluctuating water level, a composition of concrete has been selected based on the same aggregates and cement.

3. The watertightness of concrete for internal structures has been investigated.

4. Devices measuring the amount of heat release in cement and concrete have been designed, manufactured and installed.

5. The amount of heat release during hardening of slag-portland cement and of portland cement from the Krasnoyarsk plant has been determined.

6. The maximum increase in temperature of concrete of selected mixes under adiabatic conditions has been calculated.

II. P.I. Vasil'ev of the Laboratory for Engineering Structures, has obtained, in this study of the thermal conditions of the Krasnoyarsk HEP, a preliminary estimate of the velocity of cooling of the dam, and established the time schedule for grouting the longitudinal construction joints. He also determined the distances between longitudinal construction joints and studied the problem of artificial cooling of concrete masonry.

In 1958 research has been conducted at the concrete laboratory on the resistance to cavitation of concretes of various compositions.

This investigation permits certain conclusions on the technology of concrete exposed to cavitation.

The results of this study are being applied in the design and will subsequently be used in the execution of buildings.

TECHNICAL ASSISTANCE IN SELECTION OF CONCRETE FOR THE BUKHTARMA HEP

Responsible for Research: R. E. Litvinova, Candidate of Chemical Sciences, Senior Research Worker

Since 1956 the concrete laboratory of the VNIIG renders technical assistance to the management of the Bukhtarma HEP by solving specific problems of design and selection of concretes as well as through quality control of workmanship and materials.

In 1958 the experimental placing of stiff concrete of a composition developed in 1956 at the VNIIG laboratory has been supervised by the VNIIG laboratory for concrete. Through close observation of concrete placing in field and careful study of the experimental data obtained at the VNIIG laboratory, it has been possible to develop, in collaboration with Lengidep, technical specifications for mix composition and placing of stiff concretes for the internal zones of the Bukhtarma HEP structures. According to VNIIG specifications, the cement ratio of this concrete mix was 180 kg/m^3.

On demand from the Lengidep, the laboratory also studied the selection of a concrete mix for lining the elbows of the outlet tunnel from the second chamber of the navigation lock of the Bukhtarma HEP. To ensure reliable protection from cavitation effects and to reinforce the tunnel elbows, it has been decided to use concrete grade 300, with watertightness class W-8, prepared from portland cement grade 500, with coarse aggregates having a maximum size of 50 mm.

The laboratory has also studied the problem of artificial cooling of concrete mixes, by studying [foreign] literature, as there are no data available in the Soviet dam-building practice.

A literature survey has been prepared in 1958 on the experience in cooling concrete mix of dams abroad, and apparatus were designed for the study of exothermic effects appearing during hardening of cement and concrete.

THE PROBLEM OF THE COMPOSITION OF CONCRETE PREPARED FROM LOCALLY AVAILABLE AGGREGATES AND USED FOR THE DNEPRODZERZHINSK HEP, AND EXPERIMENTAL INVESTIGATION OF PROBLEMS OF PREPARATION AND PLACING OF CONCRETE ON THE SITE

Responsible for Research: R. E. Litvinova, Candidate of Chemical Sciences, Senior Research Worker

In 1958 the main work on this subject consisted in adjusting concrete mixes made of new materials received by the concrete laboratory, and in technical assistance on the site in the design and proportioning of concretes of various grades.

The laboratory prepared concrete mixes with three different binders (portland cement, pozzolana-portland cement and slag-portland cement) for two different size gradings of sand.

Despite the unsatisfactory size gradation of the tested river sand, containing a larger amount of small particles than is required by the standard, it was possible to design suitable concrete mixes by a careful selection of aggregates and plasticizers for structures operating in areas of fluctuating water level and in submerged areas; the cement content of these mixes was 280 and 260 kg/m^3 of concrete, respectively, and thus did not exceed the limits permitted by the MES specifications.

In 1958 assistance was given to designers and contractors in various problems connected with the use of locally available cements.

On the basis of additional data, as for example those referring to the streamflow-regulation conditions in the daily variations of the tailwater level, the problem has been studied of the possible use, for certain more important parts of the structure, of pozzolana-portland and slag-portland cements available at the site instead of portland cement which could not be obtained in the required amounts.

Tests of different concrete specimens for frost resistance proved that, for concretes of grades not higher than "MRZ 100", slag-portland cement of 400 kg/cm^3 strength obtained from the Dneprodzerzhinsk plant can be used. Use of pozzolana-portland cement for concrete of this grade would require very large amounts of cement.

The laboratory also studied the influence of waste-sand aggregates, containing 2-4% of clay particles, on concrete shrinkage. This sand has been obtained from a kaolin plant.

It has been proved that use of sand containing small amounts of kaolin does not cause an increase in the shrinkage of concrete after an age of one year.

Comparative tests of concrete mixes have shown that presence of quarry fines in the crushed stone in amounts of 4 to 6% makes it necessary to increase the cement content approximately by 10 kg/m^3 of concrete; the laboratory therefore suggested to wash the crushed stone [before its use].

ADDITION OF COOLED-STONE AGGREGATES TO THE CONCRETE USED FOR THE HYDRO STRUCTURES OF THE MAMAKAN HEP

Responsible for Research: M. K. Smerdov, Senior Engineer

The study involved a visual examination of the strength of bond between the concrete and the incorporated rounded stones 150-300 mm large, cooled to -25° C. The volume of the stone added amounted to 25-30% of the total volume of concrete.

In case of favorable results it had been intended to incorporate such stones into the concrete mixes for the structures of the Mamakan HEP.

The following conclusions can be drawn as a result of these investigations:

1. When stones cooled to -25° C are added to a concrete mix having a temperature of +5° C, moisture is transferred from the concrete mix toward the cooled stone owing to the temperature difference between the concrete mix and the stones, with a resulting increase in the water-cement ratio of the concrete mix adjoining the stones.

2. Around the stones cooled to -25° C a frozen layer of cement paste is formed with a water content higher than that of the remaining mix.

3. The low temperature of the stones (-25° C) causes further freezing of the concrete mix along the contour of the stones, seriously disturbing the concrete's structure.

4. Addition of deep-cooled (-25° C) rounded stones in an amount of 25% of the total concrete volume, to a mix having an initial temperature of +5° C, causes the temperature of the mix to drop to -1.9° C.

5. On the strength of the experimental results it may be concluded that when the temperature of a concrete mix is +5° C or slightly lower addition of large stones, cooled to -25° C, causes serious damage to concrete, deteriorates its technical properties and is therefore unadvisable.

INTRODUCTION OF CORRECTIONS INTO TECHNICAL SPECIFICATIONS AND NORMS

Responsible for Research: Professor V. V. Stol'nikov, Doctor of Technical Sciences
V. V. Kind, Candidate of Technical Sciences, Senior Research Worker
Ts. G. Ginzburg, Candidate of Technical Sciences, Senior Research Worker

The purpose of this work was to prepare a final draft of GOST for methods of testing hydraulic concretes (intended to replace the existing GOST 4800-49) and a final edition of instructions on methods of design and proportioning of hydraulic concretes to replace GOST 4801-49. Both drafts were submitted to the Gosstroi U. S. S. R. for final approval.

The work consisted in a critical analysis of the notes and suggestions made by the specialists from the Technical Administration of the MES and from the Department for Standards and Norms of the Gosstroi, preparation of necessary corrections in the above-mentioned documents, discussion of the new drafts of Gost and instructions within the framework of the MES and Gosstroi.

As a result of work done in 1958, new drafts of standards and instructions were compiled and approved by the MES and the Department for Standards of the Gosstroi.

VNIIG IMENI B. E. VEDENEEV LABORATORY OF WATERPROOFING

Head: Professor P. D. Glebov, Doctor of Technical Sciences, Honored Scientist of the R. S. F. S. R.

THE PROPERTIES OF GLASS PLASTICATE* AND RUBBER AND THEIR USE FOR EXPANSION-JOINT FILLERS

Responsible for Research: N. F. Shchavelev, Candidate of Technical Sciences, Senior Research Worker

Research by: A. N. Khudyakov, Junior Research Worker

The purpose of this work was to investigate the properties of glass plasticate developed by Eng. A. S. Voevodskii for use as a filler in expansion joints of hydro structures.

Glass plasticate consists of two adjoining P. V. C. plastic sheets in between which grade TSF glass fiber is inserted. Compactness is achieved by pressing the sheets at 150 atm at a temperature of 150°C. The over-all thickness of the glass-plasticate sheets is 2.5 to 3.0 mm; they are manfactured by the Okhta Integrated Chemical Works in Leningrad and supplied in sizes of 500×600 mm.

The investigation schedule involved:

1) the study of physicochemical properties of glass plasticate;

2) compilation of technical specifications for its use;

3) the use of experimental filler, made of glass plasticate and rubber, at one of the hydro structures of the Kremenchug hydro development under erection.

The investigation was of a short duration, attention being paid mainly to: a) tests of glass plasticate for tensile strength by gradual or stepwise applied load, b) determination of resistance to repeated bending, and c) evaluation of the influence of surroundings on the mechanical strength of glass fiber and glass plasticate.

Both the glass fiber and the glass plasticate were tested for tensile strength in a press with deformation uniformly increased by 3 mm per min. The test specimens were strips 5 cm wide and 10 cm long clamped between grips. During rupture load and elongation were recorded. The maximum tensile breaking strength of the glass-plasticate specimen was 378 kg, or 75,9 kg. cm at the warp fiber, and 231 kg, or 46.2 kg/cm, at the weft fiber. Tests with loads increasing stepwise were conducted at temperatures between -10° C and +40° C, with damping of deformations after each step. The load applied varied between 20 and 50 kg per step.

From the test results it was found that the strength of glass plasticate, subjected to stepwise loading, decreases by more than one half its former value.

The influence of surroundings has been tested by keeping glass-plasticate specimens for five months in air, in water, in 1% aqueous solutions of NaOH, Na_2SO, HCl, NaCl, and in kerosene, and by testing it afterward for mechanical strength at gradually increased loads.

* [Russian term for a glass-fiber reinforced laminated plastic.]

These experiments have shown that:

1. Glass plasticate swells in water and aqueous solutions of salts, acids, and alkalies, but does not swell in kerosene. When kept in kerosene it suffers loss of weight.

2. The strength of glass plasticate depends on the effect of surroundings on the glass fiber. Glass plasticate has a satisfactory strength when kept in air. When preserved in water it loses 80% of its initial strength; when kept in NaOH solutions, its strength is reduced to zero.

The data obtained allow the following conclusions:

1. Glass plasticate cannot be used for fillers in expansion joints wider than 2 cm and provided at hydro structures erected on soft foundations.

2. Glass plasticate cannot be used for joint fillers in structures exposed to a pressure head higher than 25 m.

Parallel with the investigation of the physicochemical properties of glass fibers and glass plasticates, the waterproofing laboratory developed a design for fillers and technical specifications for their construction.

These designs have been recommended to the Ukrgidep and the construction management of the Kremenchug HEP for a trial installation at one of the structures of the Kremenchug development being erected.

Joint fillers made of sheet and structural rubber have also been developed for comparison with fillers made of glass plasticate.

Ukrgidep and the management of the Kremenchug HEP decided to arrange in 1959 experimental fillers of glass plasticate and rubber in expansion joints of the right wall of the lock.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR CONSTRUCTION WORK

Head: T. E. Kartelev, Candidate of Technical Sciences, Senior Research Worker

DESIGN OF SUITABLE COMPOSITIONS FOR PRESSURE-INJECTED CEMENT GROUTS TO WHICH NEWLY DESIGNED HIGHLY EFFICIENT ADMIXTURES HAVE BEEN ADDED

Responsible for Research: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

Research by: L. N. Paronyan, Junior Research Worker

When driving tunnels by the shield method, usually free spaces remain between the tunnel liners and the rock. At present, in all the constructions of the Lenmetrostroi, these spaces are filled up with cement-clay mortars with a clay content equal to that of cement.

This work deals with the addition, apart from plasticizing, water-retaining and repelling agents, of phosphate-electrolyte admixtures. The influence was investigated of trisodium phosphate on various properties of the cement mortar, and stone cement (matrix). An admixture of trisodium phosphate in low concentrations increases the density and strength of the cement stone. Trisodium phosphate plasticizes both clay and cement, permitting therefore a lower water-cement ratio with a resulting increase in strength of concrete. Admixture of trisodium phosphate reduces the permeability for water of the hardening mortar, especially in the initial stages of hardening. The study determines the optimum concentration of the trisodium-phosphate admixtures which depends on the clay content in mortar.

Other phosphate admixtures were also investigated.

An admixture of pyrophosphate exercises an even stronger influence on cement-clay mortars; its resulting optimum concentrations are lower than those of trisodium-phosphate admixtures.

A study of hexametaphosphate admixtures shows that they also improve the properties of cement-clay mortar and cement stone.

At the same time, combined admixtures of water-retaining and water-repelling agents were investigated. For this purpose mortars with a low cement content were tested.

The tests have shown that the optimum concentration of admixtures should be higher than in previously investigated mortars where the clay content was equal to that of the cement content.

Grouting mortars were tested for plasticity, strength, density, watertightness and tendency to leaching out. Tests were conducted on mortars at hardening ages from one month to one year under varying conditions of curing.

The influence of running water on strength and permeability for water of the cement-clay stone has also been investigated. In addition, the corrosion resistance was studied of the cement-clay stone in MgCl, Na_2SO_4 of varying concentrations, and in water from the Neva River.

Parallel with the research work, technical assistance was given in the spring of 1958 in grouting operations at the inclined tunnel of the Lenin-Square subway station. Here cement-clay mortars were used to grout a layer of alluvial-gravel soils exhibiting intensive seepage. As a result of grouting, a reliable sealing of the soil has been achieved.

THE USE OF CEMENT-CLAY MORTARS COMBINED WITH SURFACE-ACTIVE ADMIXTURES OF WATER-REPELLING AND WATER-RETAINING AGENTS

Responsible for Research: N. A. Adamovich, Candidate of Technical Sciences, Senior Research Worker

Research by: L. N. Paronyan, Junior Research Worker

The subject of the work can be subdivided into three sections.

1. Study of technology of cement-clay mortars, preparation, layout and planning of mortar-batching units, as well as of the basic principles of design of new equipment.

2 Preparation of a second edition of the "Instructions for the Proportioning and Preparation of Cement-clay Mortars Used in Hydro Construction".

In preparing this edition, use was made of remarks and suggestions from various agencies.

3. Preparation of drawings for new equipment.

The first two problems were studied by VNIIG, the third one by the Gidroenergoproekt.

For the first of these problems the team prepared a description of a layout for central clay-cement mortar batching units used in U.S.S.R. construction practice, and one layout of a similar unit built abroad. The same chapter of the study also contains a description of the layout of a clay-mortar unit used in the oil industry, of some existing types of mortar

mixers and of their characteristics, and of a laboratory-type centrifugal mixer which is now being designed by the construction office of the institute.

The second chapter contains the second edition of the Instructions which has been compiled with due consideration of the remarks and suggestions obtained from a number of specialists.

An instruction is being compiled for grouting of fissured rocks. It is intended, first of all, for the use in grouting hydro structures, but can be also applied to other types of work.

In the final edition of the Instructions the text has been changed and new appendices have been added.

GIDROPROEKT IMENI S. Ya. ZHUK DEPARTMENT FOR RESEARCH ON BUILDING MATERIALS

Head: B. F. Mikulovich, Engineer

STRUCTURAL CHANGES IN THE CONCRETE UNDER ACTION OF ALTERNATING FREEZING AND THAWING, DRYING AND MOISTENING, AND HEAT RELEASE OF CEMENTS OF DIFFERENT COMPOSITIONS

(Investigation of durability of portland-cement concrete)

Responsible for Research: P. A. Pshenitsyn, Engineer

Research Team: B. I. Khrenov, Engineer
Z. P. Gulyaeva, Engineer

This work continues the research initiated in 1957 on the durability of concrete of different mineralogical and material composition as well as of surface-active admixtures under the action of alternated freezing and thawing, drying and moistening. The study of the resistance of concrete to various [environmental] factors permits the selection of cement suitable for various types of hydraulic concretes and allows for a saving in cement clinker.

Attention has been paid to the kinetics of heat release upon hardening as a factor favoring the decrease in thermal stresses appearing during hardening; this study permits suitable means to be choosen for the prevention of cracks in concrete structures.

The study contains information on the physicochemical characteristics of the materials investigated: on the heat release of cements of different mineralogical and material composition; on the influence of surface-active admixtures, such as spent sulfite liquor and rosin soap; on the amount of heat release; data on the conditions of hardening of concrete; on the influence of alternating freezing and thawing on the changes in the dynamic moduli of elasticity of cement mortars of different compositions; on the influence of alternating drying and moistening on structural changes in the natural cement stone (concrete); on the influence of mineral and surface-active agents on the kinetics of crystal

formation during hardening of cement; and finally on the influence of alkalies contained in cement on the opaline grains included in the concrete aggregates.

In addition to research methods, suggested by GOST, the study describes several original methods for the determination of:

a) the degree of hydration of cement, by carbonizing the hardened cement mortar;

b) the amount of heat release in hardening cement;

c) the kinetics of cristal formation in the hardening concrete by the gravimetric-thermal technique.

These investigations made it possible to determine the role of mineral and surface-active admixtures to cements in controlling the heat release in cements of different composition and to explain the influence of air-entraining admixtures on the increase in the resistance to air and to frost of concrete under varying environmental conditions; thus, they opened the way to an extensive utilization of slag-portland cements for hydro structures, and gave a plausible explanation to the action of alkalies on the opaline grains of the aggregates.

These results will be useful in preparing Technical Specifications on the composition of concrete for various hydro structures and on the proportioning of concretes for various service conditions.

IMPROVED TECHNOLOGY OF PREPARATION AND PLACING OF SEMISTIFF CONCRETE MIXES IN MASSIVE BLOCKS OF HYDRO STRUCTURES

Responsible for Research: N. A. Vtorov, Engineer

Research Team: D. P. Minin, Engineer
A. E. Birnbaum, Engineer

The purpose of this study was to reduce the construction costs through maximum saving in cement by using semistiff concretes with air-entraining admixtures, and to determine the most rational compositions of semistiff concrete, methods of preparation, transportation, placing and compacting in blocks.

The study was in the laboratory and under semi-industrial conditions on an experimental construction site.

The work involved the study of preparing, placing and compacting in blocks, various mixes without any slump or with a slump varying from several millimeters to 2 cm, manufactured with gravel or crushed-stone aggregates and with two grades of cement–portland and pozzolana–according to the Technical Specifications for the Volga HEP im. V. I. Lenin.

The mixes were prepared in a S-199 concrete mixer of 250 liter capacity. Concrete was placed into the blocks by a truck-mounted crane in special 450 liter buckets with opening bottoms (Figure 71).

Concrete was compacted in the blocks by means of I-50 and V-60 vibrators. The efficiency of the vibrators was tested on concrete mixes of various compositions (Figure 72).

FIGURE 71. Placing of concrete into blocks by truck-mounted crane in 450 liter buckets with opening bottoms

The basic physicomechanical properties [strength and specific water absorption] were determined in the following manner: 1) tests for strength, on specimens prepared from the mix being placed, or on core samples taken from the concrete structure; 2) tests for specific water absorption, by pumping water in boreholes drilled in the structure.

These investigations proved that high-quality concrete mixes giving a conus slump of < 2 cm, and prepared from air-entraining admixtures (rosin oil) in an amount of 0.02% by weight of cement with a portland-cement content of 140-150 kg/m^3, or a pozzolana-cement content of 150-170 kg/m^3 (for concrete grade "200" at the age of 180 days) can be compacted in the structure by means of high-frequency vibrators type V-60. The concrete thus obtained will satisfy the strength and density requirements for hydro structures.

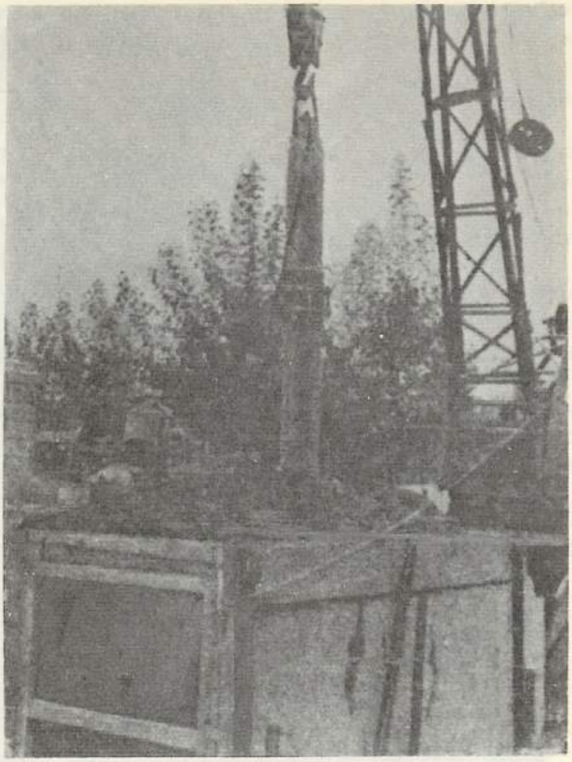


FIGURE 72. Compacting of concrete in blocks by I-50 and V-60 vibrators

TEST OF PRESTRESSED REINFORCED-CONCRETE PIPES USED IN THE CONSTRUCTION OF THE NORTHERN DONETS-DONBASS CANAL

Responsible for Research: B. F. Mikulovich, Engineer

Research by: A. D. Osipov, Engineer

In 1957-58 the design office of Gidroproekt together with the scientific research division of the latter developed the design of prestressed reinforced-concrete pipes with an internal diameter of 2.5 m and 2.8 m.

The reinforcement skeleton of each pipe run consisted of separate hoops, each made of two annular flat trusses connected by means of tie plates. Each truss had two chords (near the exterior and interior surfaces of the pipe) made of hot-rolled corrugated bars of ST-5 steel. The chords were connected by lattice work. Tests have shown that the lattices should be made of reinforcing steel grade ST-5. The assembling of hoops to form a pipe is shown in Figure 73.

Prestressing of reinforcement hoops was done by applying two equal compressive forces in the direction of the vertical diameters. After assembling

the hoops into pipe runs and concreting the pipe, the pressure was removed when the concrete attained the specified strength. The stresses in the reinforcement caused by its striving to return to its original position were transmitted to the concrete. As a result, stresses appear in the cross section of the pipe opposite in sign to those caused by the conventional service load. This made it possible to reduce the computed moments in the pipe runs.

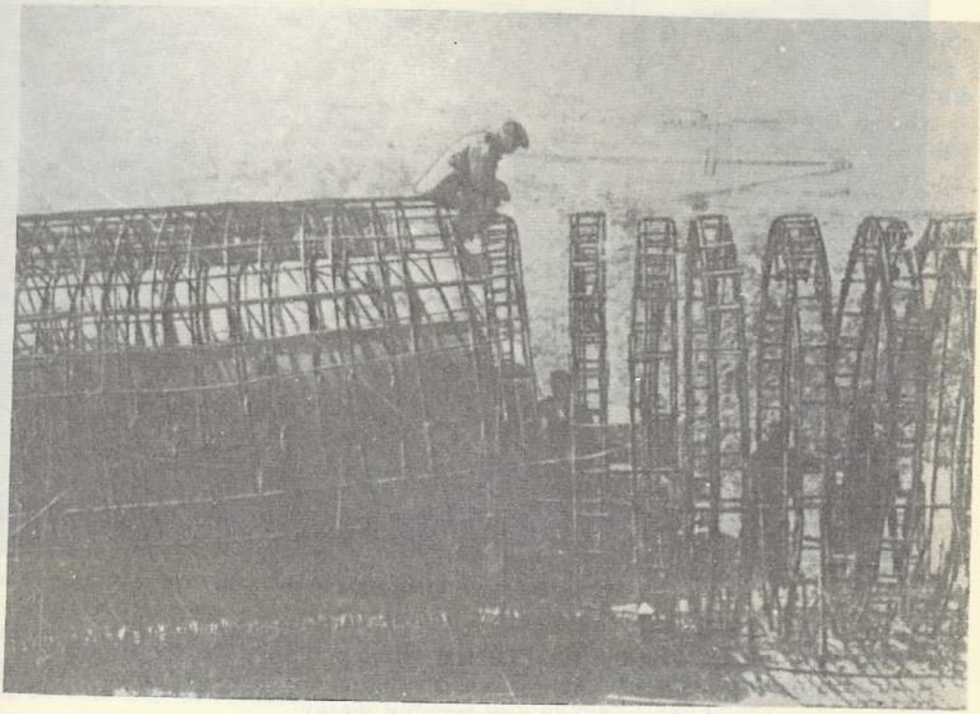


FIGURE 73. Assembly of hoops into pipe runs of the pipeline

In 1957 the laboratory of structural materials carried out tests of this pipe design and checked the possibility of using it in practice. In 1958 specialists from the Gidroproekt and the administration of the Northern Donets-Donbass Canal carried out for the first time the construction of such a pipeline.

When constructing a pipeline over the Zheleznaya gully the check of preliminary stresses in reinforcement and concrete carried out on some pipeline runs gave results nearly equal to those obtained previously in the laboratory.

The general view of the pipeline is shown in Figure 74.

The new construction is not labor-consuming and can be completed within reasonable time.

Prestressing of the pipe is done by a turnbuckle device fixed in the direction of the vertical diameters of the annular hoop, and makes it possible to

reduce the wall thickness from 45 cm to 20 cm, which results in an economy of concrete of 2.52 m^3 per pipe run, i.e. of 50%.

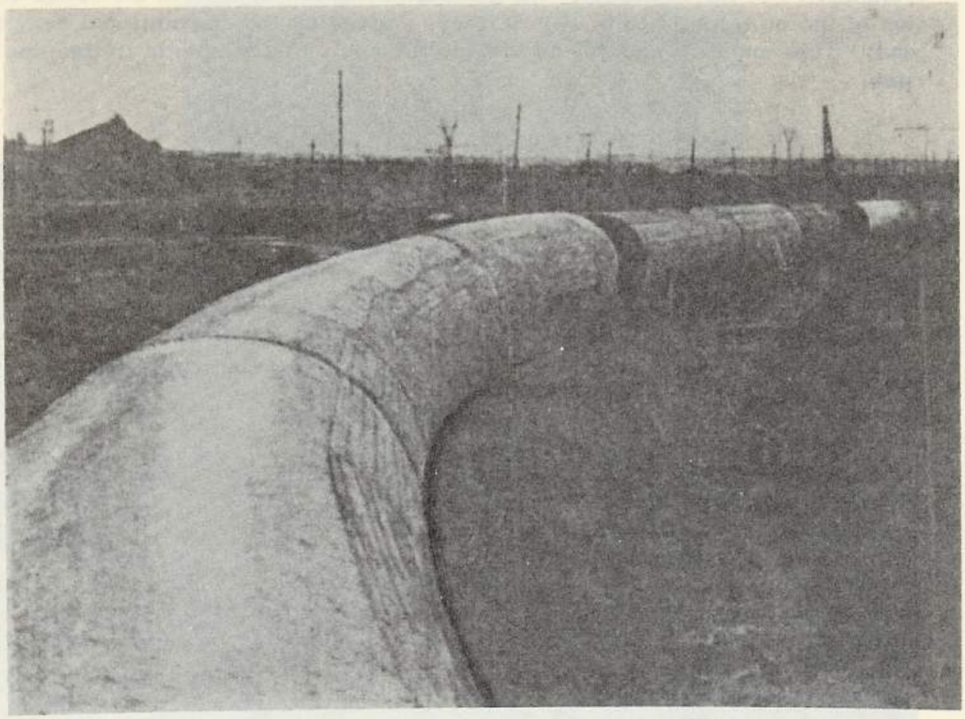


FIGURE 74. General view of the pipeline

In comparison with the first design which provided for the erection of a steel culvert, the use of a prestressed reinforced-concrete culvert resulted in an economy of steel of 0.4 t per running meter.

The construction of a trial section of the prestressed reinforced-concrete pipeline at the Northern Donets-Donbass canal proved the suitability of this type of pipes for other similar constructions.

COMPARATIVE STUDY OF FROST RESISTANCE OF HYDRAULIC CONCRETE BOTH UNDER LABORATORY AND FIELD CONDITIONS

Responsible for Research: N. A. Vtorov, Engineer

Research Team: V. D. Afonina, Engineer
A. E. Birnbaum, Engineer

The frost-resistance tests as carried out by the laboratory in accordance with GOST 4800-49 do not reproduce the actual conditions of service of concrete structures within the zone of fluctuating water level and there is no

relation between the laboratory cycles of freezing and thawing and the natural cycles.

Special frost-resistance studies carried out by the scientific research division of the Gidroproekt dealt with clearing up this problem. Because of differences in the climatic conditions of the power plants, the coefficients for conversion from laboratory to field test results are also different. This is why special stations for testing frost resistance of concrete were set up at the tailwater section of the Shirokovskaya, Skhodnenskaya, and Volga imeni Lenin power plants.

At the Shirokovskaya HEP the test specimens were placed directly on the river bank within the range of fluctuating water level; at the Skhodnenskaya HEP they were placed on metal shelves; and at the Volga HEP on a concrete bench.

Testing of concrete specimens was carried out in accordance with GOST 4800-49 but without bottom plates. The specimens were subjected to 50, 100, 200, 300, 400, 500, 600, 800, 1,000 and 1,500 cycles of alternating freezing and thawing.

Evaluation of frost resistance was done by comparing the compressive strength of specimens subjected to alternating freezing and thawing with the strength of reference specimens of equivalent age.

The equivalent age of specimens tested for frost resistance under laboratory conditions was computed for each batch and represented the over-all time of storage of specimens before freezing and the total time of thawing cycles. The instructions of GOST for determining the equivalent age were not followed, because the time of thawing of specimens was not always in accordance with the existing standards.

From laboratory and field test data, curves were plotted of changes in strength of the specimens after normal moist storage and after frost-resistance tests, In case of scatter of the strength values the mean values of strength were used for plotting the curves.

Figure 75 shows changes in strength of test specimens made of concrete No. 3 before and after tests for frost resistance. Strength of specimens tested for frost resistance under laboratory and field conditions was compared with that of reference specimens of equivalent age under normal storage conditions. The relationship involved has the form:

$$K_f = \frac{S_f}{S_{equ}},$$

where K_f = a coefficient defining the degree of frost resistance of concrete specimens [S_f = frost resistance of tested specimens; S_{equ} = equivalent strength]. Curves were then plotted of the variation in the coefficients of frost resistance for specimens tested under laboratory and field conditions, with the number of freezing and thawing cycles. These curves made it possible to determine the number of laboratory and field testing cycles showing the same frost-resistance coefficients, i.e. the same degree of failure of the specimens (Figure 76). From the data obtained, coefficients for conversion from laboratory freezing test results to field test results were calculated. Curves were then plotted for the dependence of the conversion coefficients on the number of laboratory freezing and thawing cycles (Figure 77).

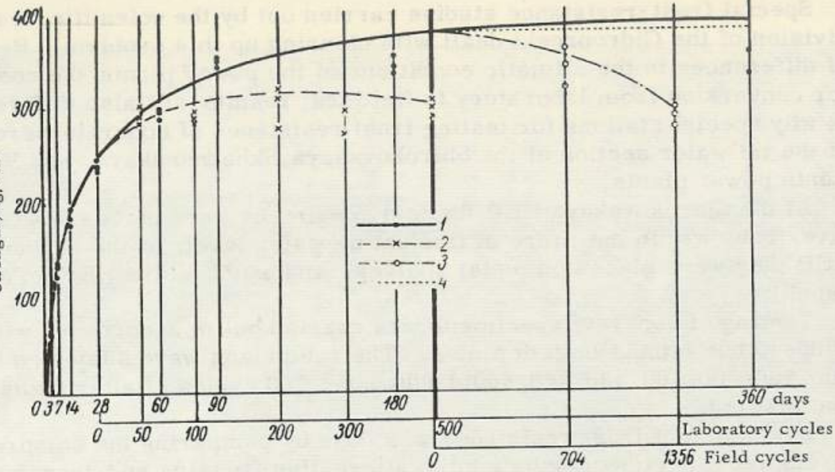


FIGURE 75. Diagram showing changes in strength of specimens of concrete No. 3 before and after frost-resistance tests

1–P_{comp} of specimens tested under normal conditions of storage; 2–P_{comp} of specimens tested in a freezer; 3–P_{comp} of specimens after testing under field conditions; 4–equivalent strength of reference specimens for comparison with strength of specimens tested under field conditions.

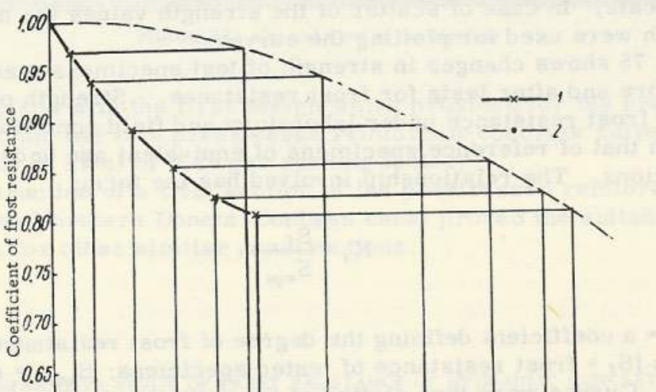


FIGURE 76. Frost-resistance coefficients for specimens of No. 3 concrete, tested under laboratory and field conditions

1–frost-resistance coefficients of specimens tested in laboratory; 2–frost-resistance coefficient of specimens tested under field conditions.

As was found from laboratory and field tests, the coefficients of conversion depend on the number of freezing-thawing cycles, but do not depend on the type of concrete mix.

At the Shirokovskaya HEP the conversion coefficient was 6.8 for 100 laboratory cycles, 4.7 for 200, 2.3 for 600, 1.6 for 1,000 and 1.3 for 1,500 cycles.

At the Skhodnenskaya HEP the specimens were subjected to 560 cycles during four winter seasons (i.e. one season less compared with the tests at the Shirokovskaya station) and did not show any reduction in strength.

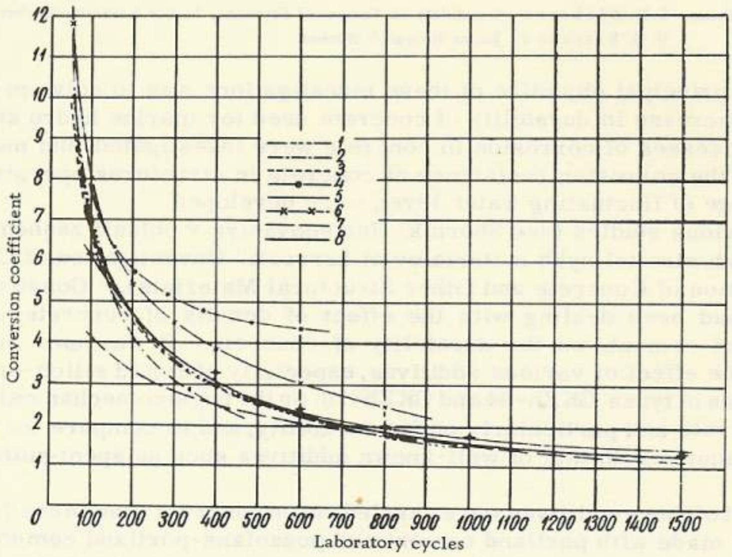


FIGURE 77. Dependence of the conversion coefficients on the number of freezing and thawing cycles carried out in the laboratory

1–for mix No. 1; 2–for mix No. 2; 3–for mix No. 3; 4–for mix No. 5; 5–for mix No. 7; 6–for mix No. 8; 7–for mix No. 10; 8–for average values of conversion coefficients.

From the test results obtained it can be assumed that the destructive action of one winter at the Shirokovskaya HEP is equal to the action of at least 4 to 5 winter seasons at the Skhodnenskaya HEP.

It was found that the specimens, kept under conditions of fluctuating water level at the Skhodnenskaya HEP, regain their initial strength in summer.

Specimens made of cement with a varying content of gaize were tested for frost resistance at the Shirokovskaya HEP. These tests showed that pozzolana-portland and portland cements, even if their content of gaize is small, are unsuitable for frost-resistant concrete mixes.

NIIZhB OF THE ASiA OF THE U.S.S.R. CENTRAL LABORATORY FOR [CONCRETE] CORROSION

Head: V. M. Medvedev, Candidate of Technical Sciences

THE EFFECT OF SILICO-ORGANIC COMPOUNDS ON THE DURABILITY OF CONCRETE USED FOR HYDRO STRUCTURES OPERATING WITHIN THE RANGE OF FLUCTUATING WATER LEVEL

Responsible for Research: Professor V. M. Moskvin, Doctor of Technical Sciences, Associate Member of the ASiA of the U.S.S.R.

Research Team: S. N. Alekseev, Candidate of Technical Sciences, Senior Research Worker
V. G. Batrakov, Junior Research Worker

The principal objective of these investigations was to solve problems of the increase in durability of concrete used for marine hydro structures. The processes of corrosion in concrete were investigated, and means to increase the corrosion resistance of concrete in structures operating within the range of fluctuating water level, were developed.

Previous studies (see Sbornik: Issledovaniya v oblasti zashchity betona i drugikh stroitel'nykh materialov ot korrozii" (Investigation of Corrosion Protection of Concrete and Other Structural Materials).- Gosstroiizdat, 1958) had been dealing with the effect of density of concrete, and of the grade of cement, on the durability of concrete. It was now important to study the effect of various additives, especially of liquid silico-organic admixtures of types GKZh-94 and GKZh-10, on the physicomechanical properties of concrete and particularly on its durability, and to compare the effect of these liquids with that of well-known additives such as spent-sulfite liquors, etc.

Test of these additives were carried out on concrete specimens 7×7×21 cm in size made with portland cement and pozzolana-portland cement.

The air entrainement of concrete mixes was ascertained by means of a special compressor-type device. Tests involved intermittent wetting and drying, passage of capillary moisture, and the checking of frost resistance in aggressive media. The state of the specimens was evaluated during the corrosion tests (without rupture) by means of a resonance-type modulus-measuring gage, according to the loss of water and on their external appearance. Tests with intermittent wetting and drying were carried out on an automatic conveyor unit.

A 5% solution of sodium sulfate was used for wetting. The frost resistance was determined by accelerated tests in salt solutions, and particularly in salt solutions simulating sea water.

Petrographic and electron-microscope research methods were used in the study of structural changes in concrete.

The study proved that liquid silico-organic additives of the GKZh-94 and GKZh-10 types [provided their amount does not exceed a maximum value which would lead to decrease in strength] could be used to increase durability and particularly frost resistance of concretes. This maximum (optimum) amount has been determined. The additives were found to have a strong plasticizing effect which, for the GKSh-94 additive, may be explained by its property to increase evolution of gases, and, for the GKZh-10 additive, by its

enhancing effect on air entrainement. These additives slow down the process of capillary absorption of solutions by the concrete mix, which may be explained by their water-repelling action. They have a higher effect on the increase in resistance of concrete to alternate wetting and drying, freezing and thawing than the well-known neutralized air-entraining rosins and spent-sulfite liquors. By using these plasticizing properties, savings of up to 10% cement may be obtained.

These additives can be recommended for concrete used in structures operating within the range of fluctuating water level and located at the Northern and Far East seas.

PART IV

HYDRAULICS

VNIIG IMENI B. E. VEDENEEV HYDRAULIC ENGINEERING LABORATORY

Head: P. A. Voinovich, Candidate of Technical Sciences, Senior Research Worker

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR DAMS AND HYDRO DEVELOPMENTS

Head: D. I. Kumin, Doctor of Technical Sciences

STREAMFLOW JUNCTION BETWEEN THE HEADWATER AND TAILWATER OF THE HIGH SPILLWAY DAMS ON ROCKY FOUNDATIONS, TAKING INTO ACCOUNT AIR ENTRAINMENT AND SCOURING AT THE TAILWATER SECTION

Research Team:
From the Laboratory for Hydraulic Engineering:
A. F. Burkov, Candidate of Technical Sciences, Senior Research Worker
N. B. Isachenko, Candidate of Technical Sciences, Senior Research Worker
G. L. Rubinshtein, Group Engineer
From the Laboratory for Dams and Hydro Developments:
A. G. Solov'eva, Candidate of Technical Sciences, Senior Research Worker
G. A. Yuditskii, Candidate of Technical Sciences, Junior Research Worker
L. G. Moskvina, Postgraduate

The investigations on the junction of headwater and tailwater streamflow at high spillway dams on rocky foundations were carried out in 1958 and they are a continuation of the work started in 1956 at the VNIIG in connection with the construction of high dams.

It was necessary to reach within the shortest possible time conclusions on which recommendations could be based. The investigations limited themselves therefore to the type of streamflow junction most typical of the given conditions in which an upturned bucket deflects the spilling jet and throws it clear of the structure.

The investigations covered the following problems:

1) recommendations for the proper choice of velocity coefficients φ;

2) investigations of the depth and shape of the scour funnel in relation to the structure of the rock foundation;

3) investigations of the actual pressure in the scour funnel and in the rock cracks;

4) preliminary recommendations for selecting the design of energy-dissipating structures suited to the flow regime of Siberian and Far Eastern rivers: these recommendations are based on earlier research work of the VNIIG, and on existing literature;

5) some directives, based on the published data, regarding the effect of air entrainment on the hydraulics of the junction between the head- and the tailwater.

As regards the first problem, the investigation covered the systematization of data on head losses at the spillway face of high spillway dams. Recommendations were made regarding the selection of the velocity coefficient φ in relation to the height of dam, the spillway discharge, the radius of the upturned bucket, and the condition of the spillway surface. As a result of these investigations, the velocity coefficient φ was found to vary within admissible limits from 0.6 to 0.95.

As regards the second problem, local scouring of the rock foundation was studied on a foundation made of nonbonded stone blocks of different shape, as well as on a foundation composed of rounded fragmentated rocks.

The following results were obtained:

1. Approximate formulas for the depth and the length of the local scouring funnel:

For nonbonded stone blocks:

a) depth of scouring funnel

$$h_s = t\left\{3.7\,\varepsilon\left(\varphi^2\frac{T_0}{d}\right)^{3/4}\frac{1}{\mathrm{Fr}_{\text{contr.}}}\left[\varphi^2\frac{z_0}{t}-\left(1-\varphi^2\right)\right]^{1,3}\right\},$$

b) length of scouring funnel

$$l_s = \frac{1}{\sqrt{\mathrm{Fr}_{\text{contr.}}}}\left\{3.0+3.6\,\lambda\left(\varphi^2\frac{T_0}{d}\right)^{1/4}\left[\varphi^2\frac{z_0}{t}-\left(1-\varphi^2\right)\right]\right\}$$

For fragmentated rounded rocks the depth of scouring funnel equals:

$$h_s = \frac{\left(0.11+0.49\sqrt[4]{\mathrm{Fr}_{\text{up}}}\right)\sqrt{q\sqrt{zt}}}{\sqrt[4]{qd}}.$$

where
- h_s = depth of scouring funnel;
- t = depth of tailwater;
- l_s = length of scouring funnel;
- T_0 = head (including velocity head) of the headwater above the bottom of the tailwater channel;
- z = head drop between headwater and tailwater;
- z_0 = head drop between headwater and tailwater with allowance for the velocity head;
- q = specific discharge at the downstream apron;
- d = diameter of sphere having the same volume as the individual stone blocks;
- $\mathrm{Fr}_{\text{contr.}}$ = Froude number at the level of the tailwater bottom (related to the contracted section);
- Fr_{up} = Froude number at the upturned bucket;
- ε and λ = correction coefficients allowing for the inclination of the nappe, the arrangement of the rock layers, the shape of individual blocks, etc.

2. The values of the above coefficients were determined.

3. Certain characteristic features of the mechanism of rock-foundation erosion were established.

As regards the third problem, the investigations studied the actual pressures at the channel bottom in the region where the falling nappe is diverted by the upturned bucket, and established the following:

1. The maximum value of the gradient of the averaged pressures.

2. The maximum amplitude of pressure fluctuations occurs in the region of maximum pressure, and may be found from the formula

$$Q_{\max} = \frac{10.5\, z_0}{h_2/h_s\ ^{2.23}},$$

where $h_2 = t + h_s$ is the depth of the tailwater + the depth of scouring funnel.

As regards the fourth problem, on the basis of field investigations carried out earlier by VNIIG, recommendations were made for the design of spcial energy-dissipating structures at the tailwater section or, alternatively, for diverting the falling nappe clear of the structures. The solution will, in each case, depend on the character of the foundation, the discharge, the height of the structure, etc.

As regards the fifth problem, on the basis of the systematization and the critical analysis of data in the existing literature, the following recommendations were made:

1. Depth of stream with entrained air

$$h_a = h\left(1 + \frac{w_a}{w_w}\right),$$

where h = depth of the stream without entrained air;
$\frac{w_a}{w_w}$ = ratio of air volume to water volume;

$$\frac{w_a}{w_w} = k\sqrt{\mathrm{Fr} - 45},$$

$$\mathrm{Fr} = \frac{v^2}{gR},$$

v and R = flow velocity and hydraulic radius, respectively, for a stream without entrained air
k = 0.060 to 0.095 (depends on the condition of the spillway surface).

2. For an air-entraining stream, the frictional resistance at the channel walls is smaller than in a stream without entrained air, while the resistance at the air-water interface is greater. As a result, the over-all energy losses in air-entraining streams are about the same as for streams without air entrainment. It is therefore recommended that, for the time being, when calculating energy losses in air-entraining streams, the same roughness coefficient be taken as in the case of streams without air entrainment.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR HYDRAULIC ENGINEERING

Head: P. A. Voinovich, Candidate of Technical Sciences, Senior Research Worker

SELECTION OF SUITABLE TYPES OF ENERGY DISSIPATORS ACCORDING TO THE EROSIVENESS OF THE WATER STREAM, UNDER CONDITIONS OF A TWO-DIMENSIONAL AND A THREE-DIMENSIONAL PROBLEM. DIRECTIVES REGARDING THE OPERATION OF GATES, AND WAYS OF REDUCING LOCAL SCOUR IN THE TAILWATER SECTION

(Selection of suitable types of energy dissipators according to the water-stream erosiveness, under conditions of the two-dimensional problem. Directives regarding the operation of spillway-dam gates).

Responsible for Research: F. G. Gun'ko, Candidate of Technical Sciences, Senior Research Worker

Research by: V. E. Lyapin, Senior Engineer

Many types of energy dissipators below spillways are presently in use, operating mostly under three-dimensional conditions. However, it has not yet been established which type is the most effective for a given set of conditions.

Laboratory investigation of spillway structures of large hydro developments proved that a sufficient degree of energy dissipation can be obtained by one or two rows of baffles placed on the spillway apron close to the contracted section of the nappe (see, e. g. the Kuibyshev, Stalingrad, Gor'kii, Novosibirsk, and Mingechaur HEPs). However, experience at the Kuibyshev and Novosibirsk HEPs and also at American hydro developments (Bonneville, McNary, etc.) showed that these baffles are subject to cavitation, and they cannot therefore be recommended without reservation.

On the other hand, laboratory and field tests on auxiliary dams have proved that these simple structures are practically free from cavitation. The same can be said of stilling pools and combined types of energy dissipators.

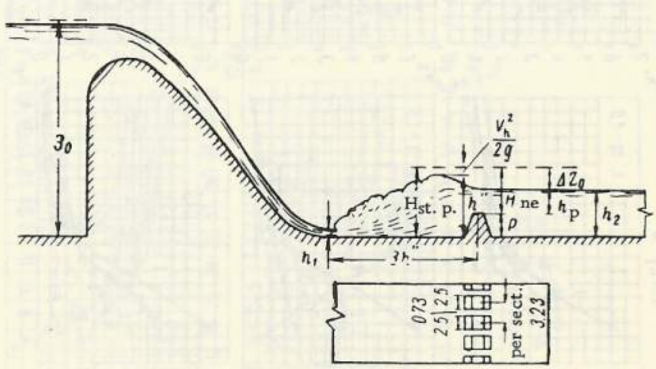


FIGURE 78. Discontinuous water-deflecting wall

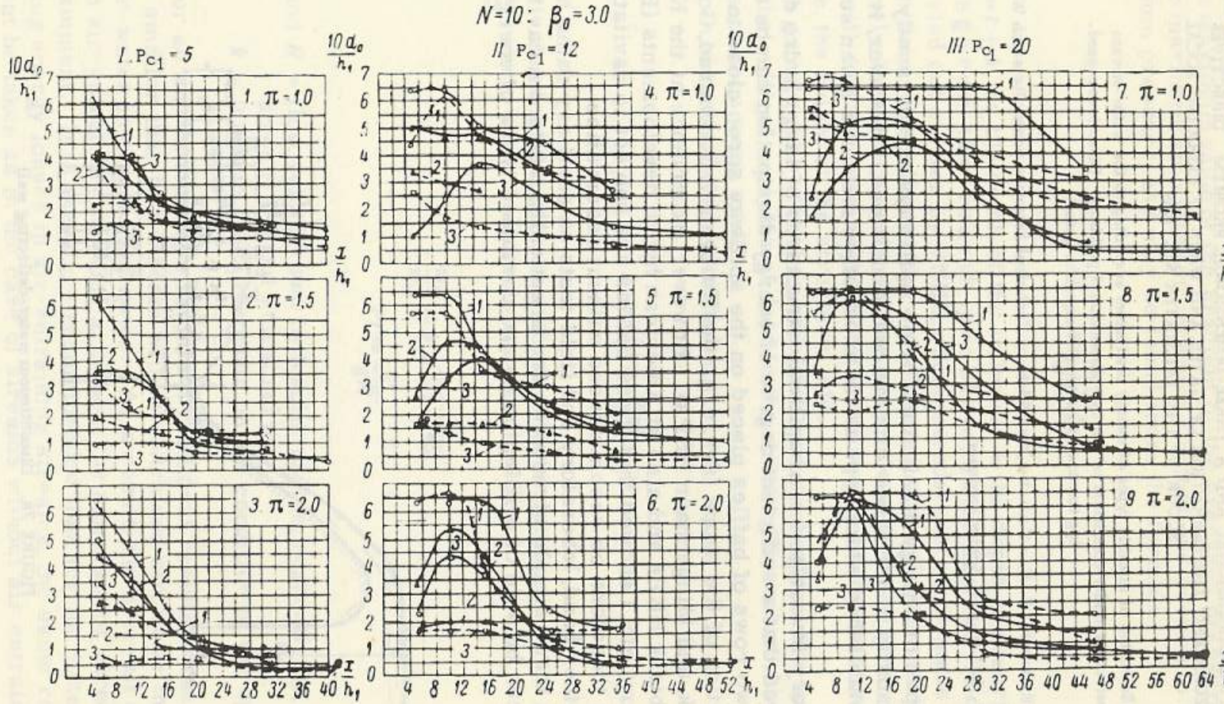


FIGURE 79. Curves of variation of relative critical diameters (function of the relative distance between the given stream cross-section and the beginning of the apron).

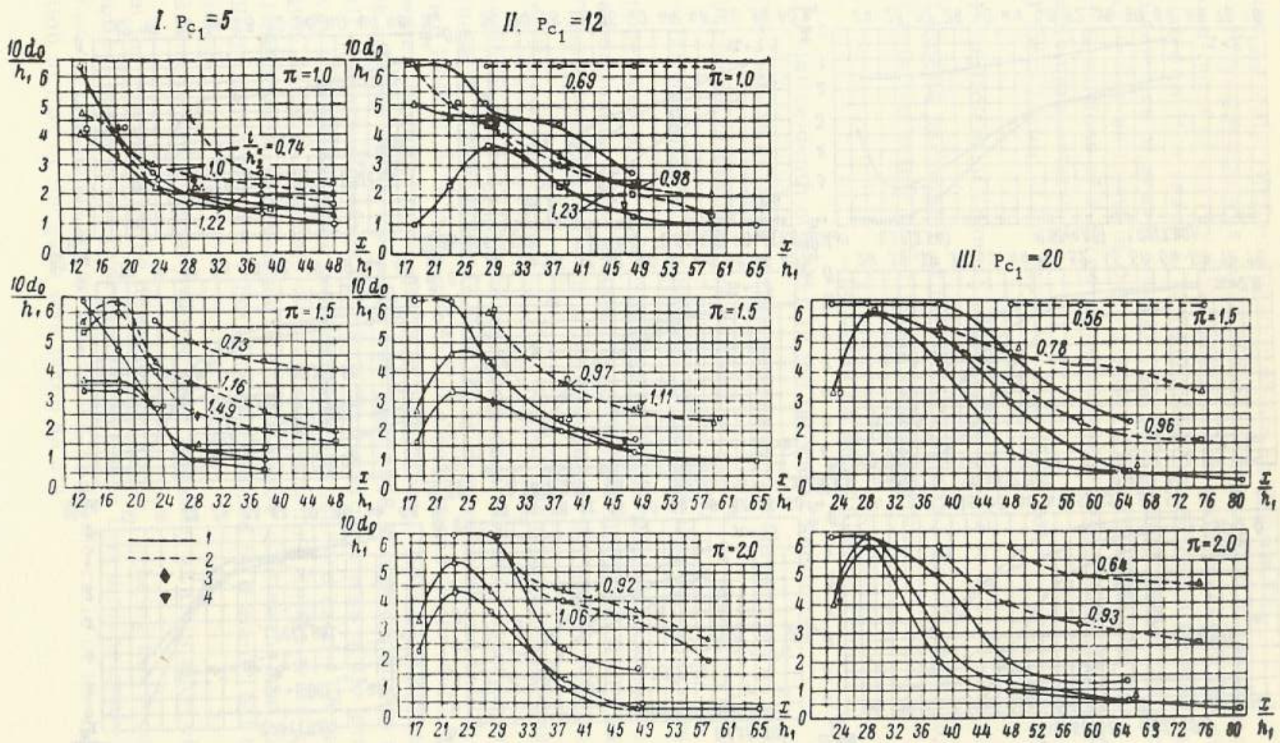


FIGURE 80. The function $\frac{10d_0}{h_1} = f\left(\frac{x}{h_1}\right)$ for an auxiliary dam, for conditions of the two-dimensional problem for $P_{c_1} = 5, 12$, and 20 (dotted line) and for conditions of the three-dimensional problem for $N = 10$ and $\beta_0 = 3.0$

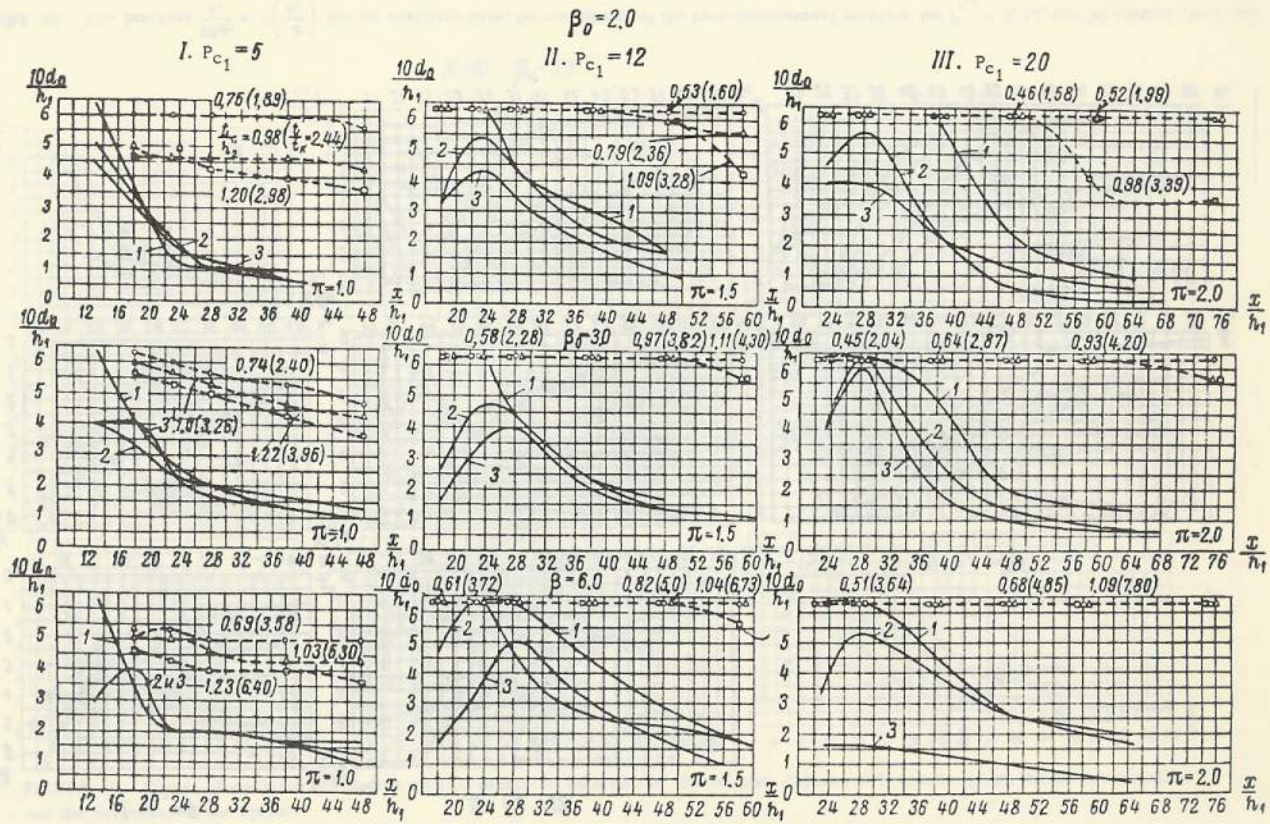


FIGURE 81. Graphs showing the relationship $\frac{10d_0}{h_1} = f\left(\frac{x}{h_1}\right)$ for a smooth water deflector for the bottom-flow conditions (dotted line) and for a water-deflecting wall with $N = 10$, $\beta_0 = 2, 3, 6$, varying P_{c_1} and water depth

D. I. Kumin has proposed energy-dissipators of a new type, not subject to cavitation, and this has been adopted by the Gidep (see annotations to the investigation work of VNIIG, finished in 1957). Laboratory and field tests (e. g. at the Kakhovka dam) also confirm the satisfactory operation of a discontinuous water-deflecting wall shown in Figure 78.

An investigation was made of the erosive action of the water stream on various types of energy dissipators. The erosiveness of the stream was evaluated by Professor A. N. Rakhmanov's critical-diameter method, i. e. the diameters of gravel and sand particles which resist the scouring action in particular sections of the stream. The investigations were carried out under conditions of a three-dimensional problem adopting the following variable parameters:

a) form factor of the nappe $N = \frac{b}{h_1}$ (5 to 20),

b) coefficient of channel widening $\beta = \frac{B}{b}$ (2 to 3.6),

c) kinetic parameter of the tailwater end of the nappe P_{c_1} (5, 12, 20),

d) relative height of the water-deflecting wall

$$\pi = \left(\frac{p}{h_1} = 1.0;\ 1.5 \text{ and } 2.0\right) \text{etc.}$$

e) relative depth of water in the tailwater section $\frac{t}{p}$.

The results of these investigations were represented as curves showing the variation of the relative critical sizes $\frac{d_0}{h_1}$ (or more correctly $\frac{10\,d_0}{h_1}$ with the relative distance $\frac{x}{h_1}$. between the respective stream cross-section and the upper end of the water-deflecting wall. Figure 79 shows such a graph for a continuous and a discontinuous (dotted lines) auxiliary dam.

Figure 80 illustrates the effectiveness of the water-deflecting wall by giving the curves $\frac{10\,d_0}{h_1} = f\left(\frac{x}{h_1}\right)$ for three-dimensional conditions, both with and without (dotted line) the auxiliary dam. Figure 81 represents the erosiveness of a stream under both three-dimensional and two-dimensional (dotted line) flow in the presence of a continuous water-deflecting wall, the tailwater depth being the same in both cases. As a result of the investigations carried out in the years 1957 and 1958, it was found that for relatively small channel widenings ($\beta_0 = 2$ to 3), the erosiveness of the stream below the energy dissipators does not depend to any marked extent on the type of dissipator while, in channels with marked widening (in the tests β_0 was equal to 6), the lowest erosiveness was found below continuous and discontinuous auxiliary dams, the latter being characterized by a somewhat better performance.

Both types of auxiliary dams were installed at a distance of $\lambda = \frac{3\,h_2}{h_1}$ from the beginning of the downstream apron; h_2 being the second critical (junction) depth of the hydraulic jump under two-dimensional conditions.

For discontinuous water-deflecting walls the ratio of the length of each section to that of the gaps was taken as 3:4.

CHECKING, UNDER FIELD CONDITIONS, RESULTS OF LABORATORY INVESTIGATIONS OF HYDRAULIC PHENOMENA AT HYDRO DEVELOPMENTS AND HYDRO STRUCTURES

(Field investigations on the flow regime and the conditions of the river channel at the Kama, Kakhovka, and Gor'kii HEPs)

Responsible for Research: F. G. Gun'ko, Candidate of Technical Sciences, Senior Research Worker

Research Team: P. I. Sokerin, Junior Research Worker
O. V. Maslovskii, Engineer

The Ministry of Electric Power Stations of the U. S. S. R. charged the Institute with the investigation of the causes of local scour in the river channel of some hydro developments.

Though initially, the investigations were intended to cover the Kama and Kakhovka HEPs only, in 1958 it was decided to include the Gor'kii HEP in the investigation program. In drawing the final conclusions, use was also made of data obtained at the Dubossary HEP.

The field investigations at the Kama and Gor'kii HEPs were carried out by the research department of the Mosgidep, and at the Kakhovka and Dubossary HEPs by the Ukrgidep.

As a result of these investigations the following conclusions were drawn

A. Causes of local scour in the tailwater area and in the water guiding and deflecting structures

1. One of the main causes of local scour is the departure (due to various technical reasons) from the recommendations of the scientific research and design institutes regarding the regime of water discharges, particularly during the construction period. Field observations at the above hydro developments revealed cases of cracks in the spillway structures, which lead to the concentration of water currents in the tailwater section and hence to increased specific discharges and high flow velocities at the end of the downstream apron.

This effect was particularly noticeable where the recommendations for the operation of gates below the combined type of HEP were not adhered to, and where the absence of any energy dissipator or stream-guiding structures at the downstream aprons led to particularly deep scouring.

This scour, though fairly advanced by 1958, did not yet endanger the stability of the end sections of the water deflectors or the rigid aprons, since it either did not exceed the expected value, or it was possible to repair the affected portions with stone or concrete fill.

2. One of the causes of the landslides occurring at the right river bank of the Kama HEP is the absence of any bank consolidation, contrary to the recommendations of VNIIG which were incorporated in the design.

3. The riprap below the rigid aprons was lined by reinforced concrete slabs bonded crosswise and lengthwise. The lowering of the riprap due to the scouring of the river channel caused the sagging of some 2 to 3 rows of slabs, and the breakage of some of them.

4. Riprap or lining of concrete blocks showed good resistance to erosion, adapting itself to the irregular shape of the eroded channel bed, thus creating an "armored" blanket protecting the slopes of the scour funnel.

B. Checking, under field conditions, the results of hydraulic-laboratory investigations on hydro developments, as well as the correctness of recommendations based on these results

The actual operating conditions at the investigated hydro developments differed considerably from laboratory conditions. Thus, during the 1958 floods at the Kama HEP, 23 turbine units were in operation as against 18 in the laboratory tests; at the Kahkovka HEP, during the same floods, the water outlets were not operating, wheras in the laboratory model, for corresponding discharges, the outlets were in operation, etc. Moreover, there were changes both in the actual channel profiles and in the discharges. For these reasons it proved impossible to make a comprehensive comparison between the hydraulic phenomena in the prototype and in the model.

A partial comparison showed the following:

1. Due to the inertia of the luminous floats, and perhaps also to the greater effect of water viscosity in small-scale models, the surface-flow velocities below the apron were lower in the model than in the prototype.

2. The actual discharges through the outlets of the Kakhovka HEP for different openings of the gates showed almost complete agreement with the values obtained for one outlet bay of the 1:20 laboratory model.

3. The scour characteristics at the tailwater section of the Kama HEP correspond to those of maximum scour forecast by the laboratory for unfavorable conditions of water passage during construction of the HEP.

C. General conclusions

In order to avoid possible defects connected with the passage of water during construction and operation of the HEP, the following rules should be observed:

1. The institutions in charge of the construction and operation of the HEP should adhere rigorously to the recommendations of the scientific research and design institutes with regard to the flow regime and particularly the operation of the water outlets.

2. Between the rigid apron and the unprotected channel there should be a transitional section in which the channel is protected by riprap or concrete blocks, their thickness depending on the erosive action of the stream. The use of reinforced-concrete slabs for the protective lining is not recommended due to their insufficient flexibility.

3. Bank-consolidation work at the tailwater section of the hydro development should be done simultaneously with the erection of the spillway structures.

4. The hydraulics of the tailwater section should be further studied; this refers particularly to:

a) the field investigation of tailwater hydraulics and comparison of results with those obtained in laboratory research;

b) the use of models for the study of local scour (problems of theory and the use of various erodible materials in models);

c) the methods and equipment for testing the erodibility of actual soils and rocks;

d) flashy streams and measures to prevent their occurrence (dissipators used as discharge distributors, gate operation, etc.).

At all the hydro developments correct operation procedures for the passage of flood waters were established. On the basis of investigations conducted during 1958, more accurate schemes for operating the gates of the spillway dams of the Kama and Gor'kii HEPs were suggested.

FUNDAMENTALS OF CALCULATION AND DESIGN OF EARTH STRUCTURES WITH UNPROTECTED SLOPES RESISTANT TO THE ACTION OF WIND-DRIVEN WAVES

Responsible for Research: I. Ya. Popov, Senior Engineer

Modern hydro developments always involve the construction of large storage reservoirs on whose surface wind-driven waves of a height of up to 3.5 m are liable to form. It is therefore necessary to consider the problem of the protection of the earth structures (dams and dikes) and in certain cases also of the natural banks of newly formed reservoirs. Up to the present, the most common type of bank consolidation consists of concrete slabs, or stone revetment in the form of slabs or riprap. The cost of such a protection often amounts to 30 - 40% of the total cost of the structure.

For very large earth structures the cost of slope protection may amount to hundreds of millions of rubles. This is why hydraulic engineers have recently been trying to avoid the use of concrete for slope protection, and to devise new and cheaper methods for protecting the slopes against wind-driven waves. Among such new methods, a particular technique whose principle is described below deserves special attention.

In this method the slope of the earth structure exposed to the water is given a special, flattened profile, with the result that the wave energy is dissipated over a large area, and the amount of wave energy per unit area of the submerged slope is much less than in the case of steep slopes. The distribution of energy dissipation over a greater area enables the designer to reduce the weight of the slope protection and, in certain cases, to omit it altogether.

The aim of the present study was to prepare recommendations for determining the shape and the dimensions of stable, unprotected slopes exposed to wave attack, given the wave characteristics, and the grain-size distribution of the material of the earth structures.

The study started in 1954. The results of the first year were published in the form of a report which contained a brief description of the existing calculation methods for the design of the slopes of storage reservoirs, and some considerations on the scope of the investigations necessary for the solution of the problem. In the absence of a theory governing the dynamic and kinematic characteristics of wind-driven waves in shallow waters, or of a theory that would permit the simulation of the processes of scouring and silt deposition under the action of such waves, it was necessary to conduct a series of laboratory scale-model tests.

These experimental investigations consisted of four test series corresponding to four values of sand-grain sizes: d = 0.2 mm (I series); d = 2.0 mm (II series); d = 3.5 mm (III series); and d = 6.0 mm (IV series).

In the first test series the formation of stable slope profiles was investigated, the slope consisting of sand having a grain size of d = 0.2 mm. The wave height was h = 5.0 cm and the wave lengths λ = 50, 60, and 75 cm.

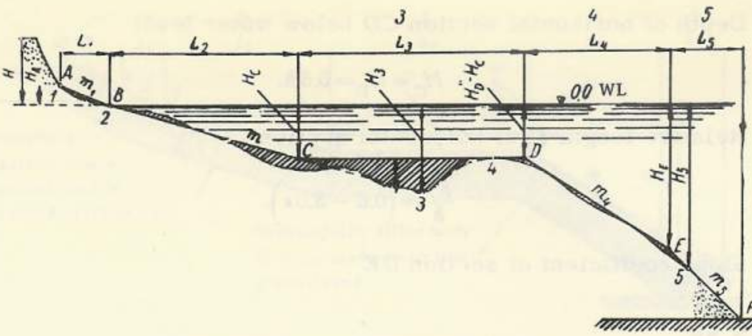


FIGURE 82. Profile of a nonprotected slope

The distribution of the water velocities at the slope surface was investigated for an equilibrium profile formed under the action of waves having a height (h) of 5.0 cm and a length (λ) of 75 cm.

Two testing units were designed. The first was installed in a glass-covered flume, 14.0 m long, 0.7 m wide, and 1.3 m high. A mechanical wave generator of the "shaking-screen" type permitted the creation of waves having a height of 20 cm and a length of 200 cm.

The second unit was mounted in a flume, 30.5 m long, 0.65 m wide, and 2.0 m high. The wave generator, of the same type as for the first unit, permitted creation of waves having a height of 40 cm and a length of 600 cm.

After analysis and systematization of the test results, the laboratory worked out a computation method for the design of a protected slope (see Figure 82). From data of measurements carried out on "equilibrium profiles" it was possible to derive the following relationships for the determination of the geometrical elements of the slope.

1. Relative height of wave moving up the slope

for $10 < \frac{h}{d} \leqslant 100$

$$\frac{H_{\text{A}}}{h} = 5.65\,\frac{d}{h} - 4.3\varepsilon - 0.58, \tag{1}$$

for $\frac{h}{d} > 100$

$$\frac{H_{\text{A}}}{h} = 0.63 - 4.3\varepsilon. \tag{2}$$

2. Slope coefficient m_1 of section AB

$$m_1 = \left(2 - 5 \cdot 10^{-4}\,\frac{\sqrt{gd}\,.\,d}{\nu}\right)\left(\frac{h}{d}\right)^{1/4}. \tag{3}$$

3. Slope coefficient m_2 of section BC

$$m_2 = (2.9 - 10\,\varepsilon)\,\left(\frac{h}{d}\right)^{1/6}. \tag{4}$$

4. Depth of horizontal section CD below water level

$$H_C = H_D = 0.6\,h. \tag{5}$$

5. Relative length L_3 of horizontal section CD

$$\frac{L_3}{h} = \left(0.6 - 3.0\varepsilon\right). \tag{6}$$

6. Slope coefficient of section DE

for $\frac{h}{d} > 40$

$$m_4 = \frac{460}{\mathrm{Re}_d} + 2.1, \tag{7}$$

for $\frac{h}{d} < 40$

$$m_4 = k\left(\frac{460}{\mathrm{Re}_d} + 2.1\right), \tag{8}$$

where k has been taken from the following table:

$\frac{h}{d}$	10	15	20	25	30	40
k	1.7	1.5	1.3	1.2	1.1	1.0

7. Relative depth at point E

$$\frac{H_E}{h} = 0.92\left[\frac{441}{\mathrm{Re}_d} + 0.6 - \left(\frac{1950}{\mathrm{Re}_d} + 2.5\right)\varepsilon\right]\left(\frac{h}{d}\right)^n, \tag{9}$$

where

$$n = 0.27 - \frac{35.9}{\mathrm{Re}_d}. \tag{10}$$

The following notations were adopted in the above formulas:

h = wave height;

ε = wave steepness;

d = grain size of soil, corresponding to 60% of the grain-size distribution curve;

ν = coefficient of kinematic viscosity of the surrounding medium;

g = acceleration of gravity

$\mathrm{Re}_d = \frac{\sqrt{gd}\cdot d}{\nu}$.

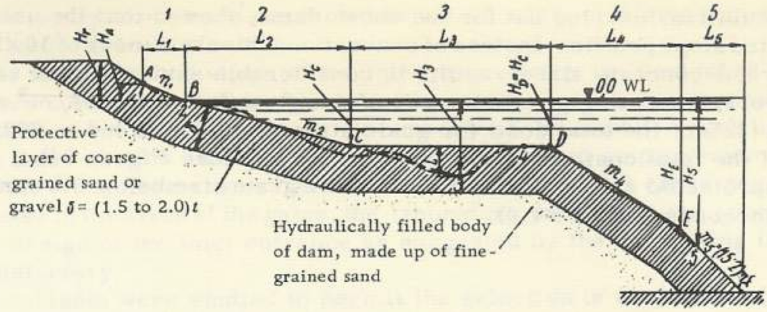


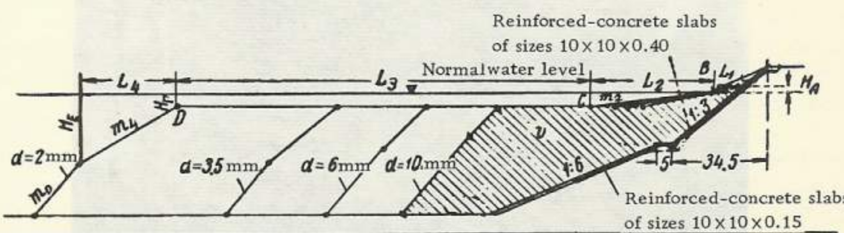
FIGURE 83. Earth dam with protective lining

In the construction of hydraulic structures made of fine-grained sand, it might be economically advisable to cover the pressure face of the slope with a protecting blanket made of coarse-grained material (see Figure 83) thus permitting reduction of the volume of earthwork. The thickness δ of the protective lining may be found from the empirical formula

$$\delta = K \cdot 0.02\left(\frac{h}{d}\right)^{1/4}, \tag{11}$$

where K is a safety factor, equal to 2.0 for $\frac{h}{d} \leqslant 70$, and 1.5 for $\frac{h}{d} > 70$.

The economical effectiveness of the earth structure with nonprotected slopes may be expressed by the dimensionless factor $d = \frac{A_1}{A_2}$ where A_1 is the cost of the additional earthwork required to form a stable unprotected slope (as against the normal dam profile) and A_2 is the cost of protecting a conventional slope by slabs, stone, etc.



Grain size of soil particles	H_A, m	m_1	L_1, m	m_2	L_2, m	H_D, m	L_3, m	m_4	H_E, m	L_4, m	m_0	v, m³/m	Cost (rubles)	Symbol
2.0	0.84	12	10	24	46	1.92	150	4.14	10.5	35.6	2.1	3321	16600	
3.5	0.84	9.75	8.18	20	38.4	1.92	100	2.98	10.5	25.2	2.1	2151	10755	
6.0	0.84	6.33	5.75	16.7	32.0	1.92	72.4	2.49	8.24	15.7	2.1	1438	7190	
10.0	0.84	3.14	2.64	14.0	26.8	1.92	52.8	2.28	6.38	10.2	2.1	381	1905	

FIGURE 84. Comparative data on protected and nonprotected pressure faces of dams Nos 1, 2, and 3 of the Votkinsk HEP

Calculations carried out for the above dams, showed that the use of nonprotected slope profiles, instead of conventional blankets made of $10 \times 10 \times 0.40$ reinforced-concrete slabs, results in considerable savings in the cost of earthwork, amounting per meter run of dam length to 5 rubles/m^3 or 3340 rubles (32% of the total cost) for grain sizes d = 6.0 mm, and to 8625 rubles (81% of the total cost) for grain sizes d = 10 mm (see Figure 84).

Nonprotected slopes made of soils with a grain size below 6.0 mm appear to be uneconomical ($d > 1.0$).

HYDRAULIC LABORATORY INVESTIGATION OF THE BOTTOM OUTLETS AT THE BRATSK HEP

Responsible for Research: A. F. Burkov, Candidate of Technical Sciences, Senior Research Worker

The main object of the investigation was:

1) to test and, if necessary, to select the contour of the sluice-gate entrance so as to ensure a smooth and nonobstructed inlet into the structure;

2) to select the shape and cross section of the gallery so as to ensure stable pressure conditions;

3) to determine the discharge capacity of the structure;

4) to study the hydraulic regime at the working and the emergency gates;

5) to study the hydraulic regime of the stream at the upturned bucket and its effect on the supports of the small concrete-conveying trestle, and to determine the conditions of flow of the water nappe diverted by the bucket into the tailwater section.

Apart from these basic problems, the present study involved the investigation of measures to be taken for the protection of the above-mentioned supports against the action of the water stream flowing from the bottom outlets.

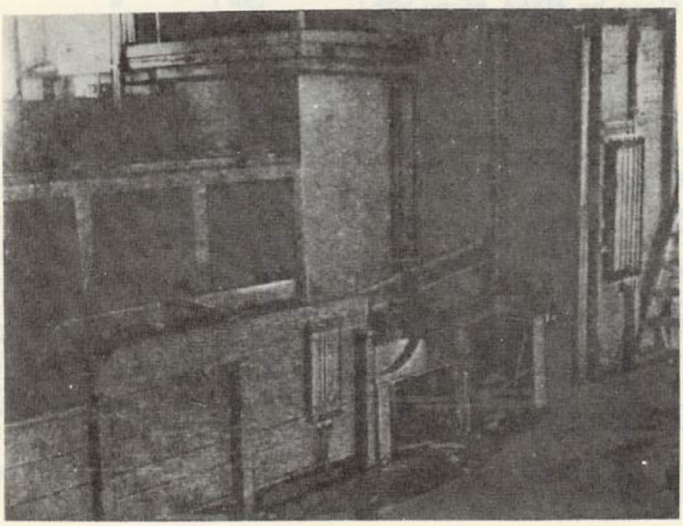


FIGURE 85. Plexiglass model of a water outlet

The investigations were conducted on a 1:35 scale plexiglass model of a water outlet (see Figure 85).

It was found that the shape of the inlet proposed by the designers does not ensure a continuous flow; however, the vacuum created is inconsiderable and does not affect the discharge capacity of the conduit. Taking into account these results and also the fact that any further improvement of the water inflow requires a considerable widening of the inlet cross section and an increase in the sizes of the gates, the laboratory arrived at the conclusion that the design of the inlet entrance, as suggested by the Projecting Institute, was satisfactory.

Two variants were studied to permit the selection of a suitable shape of the water outlet which would ensure stable flow and pressure conditions of the stream. In the first variant the height of the outlet cross section was 4.8 m, and in the second, 4.2 m. Comparison between the operation of both variants showed that the first is to be preferred, from the point of view of its discharge capacity and the degree of vacuum (which did not exceed 3.0 m W.G. at low level of the headwater section). At higher headwater levels no vacuum was formed. Therefore, in the opinion of the laboratory, the first variant may be recommended as it complies with the engineering specifications of the project.

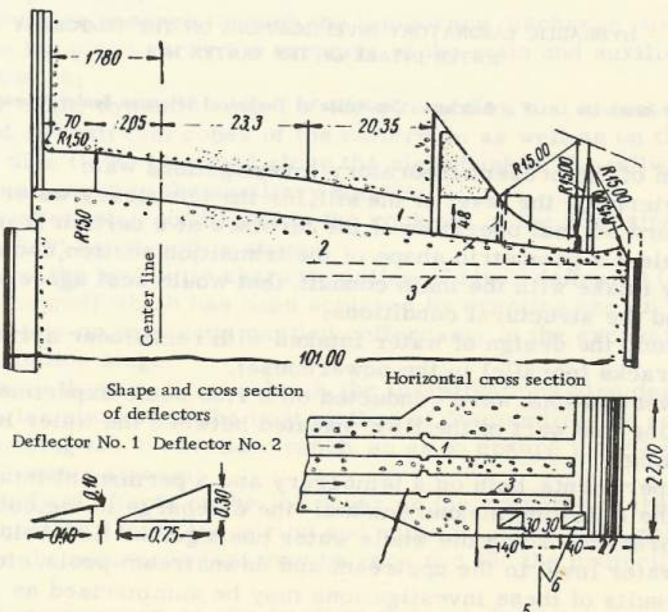


FIGURE 86. Protection of the supports of a small concrete-transporting trestle (junction of bottom water outlet with the upturned bucket at the tailwater section)

1 - deflector No. 1; 2 - ceiling of bottom (floor) galleries; 3 - deflector No. 2; 4 - recess; 5 - center line of small trestle; 6 - supports of small trestle.

The tests showed that intensive eddying is caused by the impact of the stream on the lower edge of the gate recess; this endangers the grouted-in elements of the structure and the gate seals mounted in the slot recess. In order to eliminate this effect, the laboratory suggested providing stream deflectors at the upstream faces.

The nappe issuing from the outlet gates and reaching the upturned bucket spreads fanwise, and the range of the nappe (disregarding the amount of entrapped air) is such that the scouring of the rocky river bed does not endanger the stability of the dam.

In order to study the protection of the supports of the small trestle against the action of the nappe, the laboratory made model tests of two designs of protecting structures: a sheet facing on the supports in the streamflow zone and concrete dentated water-deflecting walls as shown in Figure 86.

The sheet facing protected the relatively thin structural elements of the supports from the action of the water stream, the maximum pressure of the stream on the facing not exceeding 13 tons. The construction of dentated, concrete, deflecting walls eliminates the action of the water stream on the supports, and the laboratory therefore recommends this method.

The above results have been forwarded to the design institutions.

HYDRAULIC LABORATORY INVESTIGATIONS ON THE TEMPORARY WATER INTAKE OF THE BRATSK HEP

Responsible for Research: A. F. Burkov, Candidate of Technical Sciences, Senior Research Worker

The aim of the present laboratory investigations was:

1) to determine the level of the sill, for the temporary water intake, that would ensure normal operation of the turbines at a certain headwater level;

2) to select a geometric shape of the transition section, connecting the temporary intake with the main conduit, that would best agree with the hydraulic and the structural conditions;

3) to study the design of water intakes with rectilinear arrangement of the trash racks (parallel to the powerhouse).

The investigations were conducted on a 1:50 scale experimental model representing that part of the HEP situated between the water intake and the spiral casing.

The experiments, both on a temporary and a permanent intake structure, involved the following measurements: the discharge in the conduit, the hydrodynamic pressure along the whole water passage, the flow velocity in the conduit, the water level in the upstream and downstream pools, etc.

The results of these investigations may be summarized as follows:

1. For normal temporary operation of the turbines the level of the intake sill should be lowered by 1.5 to 2.0 as compared with the design level.

2. Since the difference in the head losses between trash racks on semicircular or rectilinear projections is negligible, while the former are more difficult to construct and to clean, the laboratory recommends the rectilinear trash-rack arrangement.

3. When running-in the first turbine units, difficulties may arise due to an excessive deposition of debris from the storage reservoir on the racks of the temporary intake. It is therefore very desirable to provide for

mechanical trash-rack cleaning not only at the permanent but also at the temporary water intake of the Bratsk HEP.

The results of these investigations were sent in the form of a technical report to the designing and constructing institutions.

SUPPLEMENTARY LABORATORY INVESTIGATIONS ON THE WATER DISCHARGE DURING THE CONSTRUCTION OF THE BRATSK HEP

Responsible for Research: A. F. Burkov, Candidate of Technical Sciences, Senior Research Worker

Research Team: V. A. Kyakk, Senior Engineer
O. V. Maslovskii, Engineer

The laboratory investigations dealt with the following basic problems:

1) determination of discharge curves of the outlet structures during the first stage of construction with completely or partly dismantled upstream and downstream cofferdams of the excavation pit;

2) study of profiles of the water table within the excavation pit of the first stage of construction;

3) study of ice passage through the temporary discharge outlets of the dam, where the piers carry the supports of the main and auxiliary construction trestlework;

4) study of the dynamic action of the streamflow and of ice on the upstream and downstream cones of the cofferdam as well as on the hydraulically filled dike (levee) erected along the right bank in the tailwater section for ensuring access to the auxiliary trestle;

5) study of erosion conditions at the rock toe of the downstream cofferdam in the first construction stage;

6) study of the hydraulic characteristics of river damming (left-bank diversion channel) which has been achieved by erecting one or two rock toes with completely or partly dismantled cofferdams in the excavation of the first construction stage.

Apart from these basic problems, the investigations also dealt with:

1) the determination of the best method of dismantling the cofferdams prior to flooding the excavation trench, so as to ensure their being carried away by the water stream;

2) the study of effects of the ice cover on the supports of the main and auxiliary trestles during the spring and autumn ice passage, and recommendations as to the measures that may be required for the protection of the supports;

3) the determination of the headwater level during a discharge $Q = 5150\ m^3/sec$ through 5 spillway bays of the spillway crest (the bottom sluiceways being completely clogged) if the tailwater level varies within 6.9m;

4) the study of the maximum possible headwater level in case of ice jamming in front of all spillway structures.

It should be noted that the scope of the research work was much wider than originally planned due, mainly, to additional requests received from the Bratsk HEP management (for a new design variant of the spillway crest, etc.).

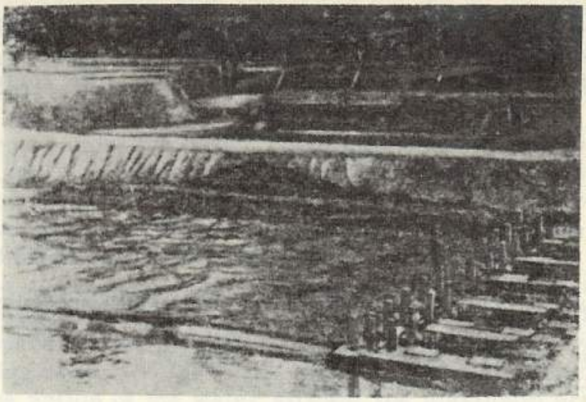


FIGURE 87. Three-dimensional 1:100 scale model of a hydro development

These investigations were conducted on a three-dimensional hydraulic model (shown in Figure 87).

The principal results of the above investigations may be summarized as follows:

1. In view of the conditions for damming the left diversion channel, it is desirable (as can be seen from the experimental discharge curves of the system "cofferdam + spillway crest + excavation") to ensure maximum dismantling of the cofferdam prior to flooding; this refers particularly to the rock toe of the downstream cofferdam (Figure 88).

2. To minimize the effect of the downstream rock toe on the discharge capacity of the above system and thus to improve hydraulic conditions of damming, the laboratory suggests not only to prolong as far as possible the action of the water stream on the rock toe during the first damming of the diversion channel, i.e. before closing the passage way in the cofferdam, but also to take effective steps for complete removal of the downstream rock toe.

3. Under field conditions, as is well known, the discharge capacity of the system may be somewhat greater than the value obtained from a laboratory model. This is due both to the scale effect and to erosion of the gravel layer over the underlying rock of the river bed.

This difference should not, however, be taken into account when calculating the damming work and it may be considered as constituting a certain reserve in the discharge capacity of the system.

4. As was found by model testing, ice passage leads to jamming in front of the spillway crest (see Figure 89). This occurs the more often, the greater the approach velocity and compactness of the ice. All other conditions being equal, the possibility of ice jamming increases if there is backwater pressure from the tailwater section or if the spillway crest is closed and the water is discharged through the bottom outlets only.

The ice pressure in front of the spillway crest may be reduced by taking suitable steps for the elimination of ice-jamming conditions both in the headwater and the tailwater sections. The ice passage through the spillway openings must be facilitated as far as possible by preventing ice clogging

at the entrance openings and by taking effective measures to eliminate ice jamming at the upstream approach section of the river. Furthermore, steps should be taken to ensure uniform ice passage during the spring and autumn ice run.

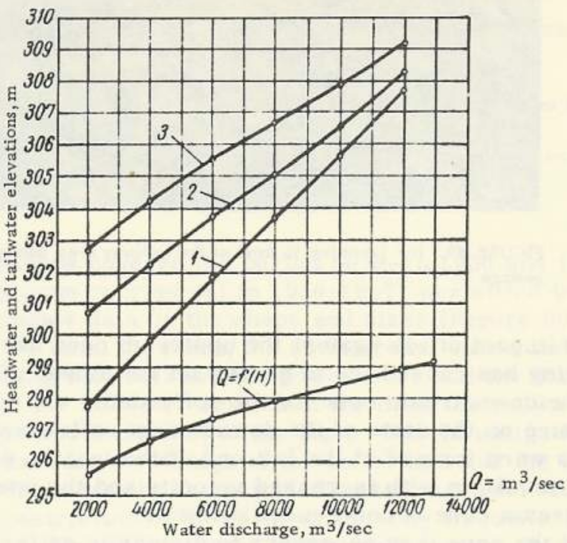


FIGURE 88. Discharge curves [Q= f(H)] for the first-stage structures at normal tailwater level

1 - cofferdams completely dismantled; 2 - cofferdams dismantled as far as elevation 297 m; 3 - idem as far as elevation 299.

REMARKS: The spillway crest consists of 11 water outlets having a width $b = 12$ m; 5 of them with a sill elevation of 290 m and 6 with a sill elevation of 292 m (the top of the piers is at an elevation of 306 m).

5. The action of ice on the supports of the small trestle was experimentally studied for two cases: at a high tailwater level (favoring ice jamming and flooding of the piers), and at the breaking-up of the ice sheet in front of the spillway crest on one or several bays.

To protect the supports of the small trestle against both cases of ice action, the laboratory recommends to embed the lower section of the steel supports of the trestle in massive concrete. The supports of the main trestle are not exposed to ice action.

6. At a discharge of $Q = 12{,}000$ m^3/sec, the maximum streamflow velocity reaches 4 to 5 m/sec at the upstream cone of the cell-type cofferdam, and from 8 to 10 m/sec at the downstream cone.

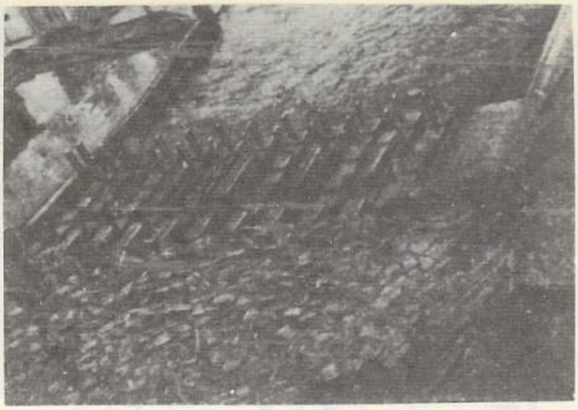


FIGURE 89. Ice jamming in front of the spillway crest during ice passage

No sudden impact of ice against the upstream cone has been observed, but ice jamming has caused ice to pile up on the slopes of the cone. The stability of the downstream cone [of the cofferdam] was found to depend in a large measure on the state of the downstream cofferdam toe. Where ice-free passages were formed at the left end of the toe, the stream, diverted in this direction, flows with increased velocity and the operating conditions of the downstream cone become more difficult.

To protect the cone it is necessary to dismantle as far as possible the central section of the toe and at the same time to consolidate the end section adjacent to the cofferdam cone. Another solution would be the construction of water deflectors at the end of the cell-type cofferdam.

7. To reduce the effect of the streamflow velocity on the river bank, the laboratory recommends to change the position of the slope as shown in the Mosgidep design, by shifting it 6 m toward the right hand bank. This will result in a marked diminution of the erosive effect of the streamflow velocity. The dike, filled with rocks of 0.5 to 1 m, will erode only at discharges (Q) close to 12,000 m^3/sec.

In addition to the measures recommended by the Mosgidep for protecting the dike against erosion, the laboratory suggests the building of settling structures made of cribwork or concrete.

8. River damming by two rock toes can be useful in the following cases: 1) to prevent the formation of a hydraulic jump beyond the first dike, 2) to reduce the streamflow velocity at the upstream passage in the diversion channel (by increasing the depth in the intermediate pool), and 3) to reduce the water discharge through the diversion channel as a result of the additional resistance of the second dike to the water flow (i.e. as a result of increasing the water level in the headwater section during the river damming).

In the present case, however, with an insufficient distance between the toes, the presence, beyond the temporary spillway crest, of a nondismantled downstream toe of the cofferdam of the first construction stage, and with flow velocities close to critical values, the positive effect of a second toe is little felt.

Therefore, the laboratory does not see any particular advantages in damming the river with two rock toes.

The results of these investigations were sent in the form of a technical report to the designing and construction institutions who made use of them.

HYDRAULIC LABORATORY INVESTIGATION OF THE STRUCTURES OF THE DNEPRODZERZHINSK HEP

Responsible for Research: F. G. Gun'ko, Candidate of Technical Sciences, Senior Research Worker

Research Team: L. V. Moshkov, Candidate of Technical Sciences, Junior Research Worker
P. I. Sokerin, Junior Research Worker
O. N. Smirnova, Technician (Acting Engineer)

The investigations covered the period between 1956 and 1958.

The investigations carried out in 1956-1957* permitted the designer to obtain more accurate data on the shape and sizes (Figure 90) of the spillway surface of the dam, the piers, the downstream apron and water deflectors, the separating wall between the powerhouse and the dam, the left-hand bank abutment at the tailwater section of the dam, the tailwater channel, as well as the downstream channel of the lock. The investigations also yielded more accurate data for the design of the cofferdam and the consolidation of its slopes, established the factors leading to increased river-bed erosion as a result of constriction of the river channel by the cofferdam and the left-bank earth dam. The proper layout of the construction channel was checked experimentally.

During 1958, the investigations dealt with a number of important problems, e. g. investigations on the discharge capacity carried out on a 1:200 scale three-dimensional model of the hydro development, etc. These investigations permitted a marked reduction in the sizes of the left-bank abutment wing, projecting into the headwater section. Studies were made, on the same model, of the problem of stripping the rock layer beyond the powerhouse, as well as of the nonsteady-flow conditions at the tailwater section for cases of sudden load fluctuations at the HEP.

The stability of the apron slabs was studied on a 1:80 scale model of three dam bays; the study permitted a considerable reduction (by about 1 m) of the slab thickness as compared with the design data.

As can be seen, the investigations conducted during the years of 1956-1958 permitted not only a considerable improvement in the hydraulic characteristics of the hydro development, but also a reduction in the amount of concrete and earthwork.

This reduction can be seen from the following figures:

1) concrete placed – 55,000 m^3; .

2) rock excavated – 55,000 m^3.

Furthermore, in 1958, the laboratory investigated, on a three-dimensional model, the design of a pier (combined type) powerhouse for the Dneprodzerzhinsk HEP with bottom sluiceways (Figure 91).

* Annotatsii zakonchennykh v 1957 g. nauchno-issledovatel'skikh rabot (Annotations of the Scientific Research Works Completed in 1957) – Gosenergoizdat. 1958.

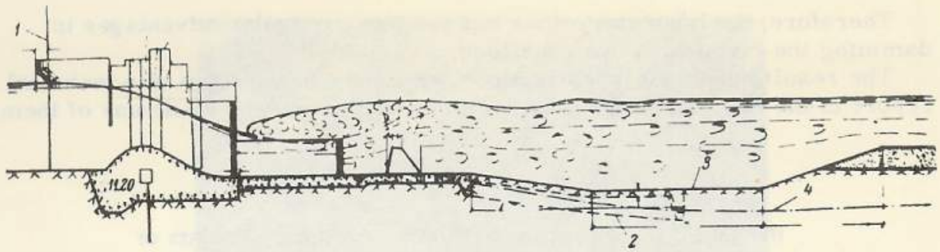


FIGURE 90. Spillway dam of the Dneprodzerzhinsk (longitudinal section)

(1) - center line of structure; (2) - cut-off profile along the bevelled edge of the downstream apron; (3) - stripped surface; (4) - surface of stable eroded area.

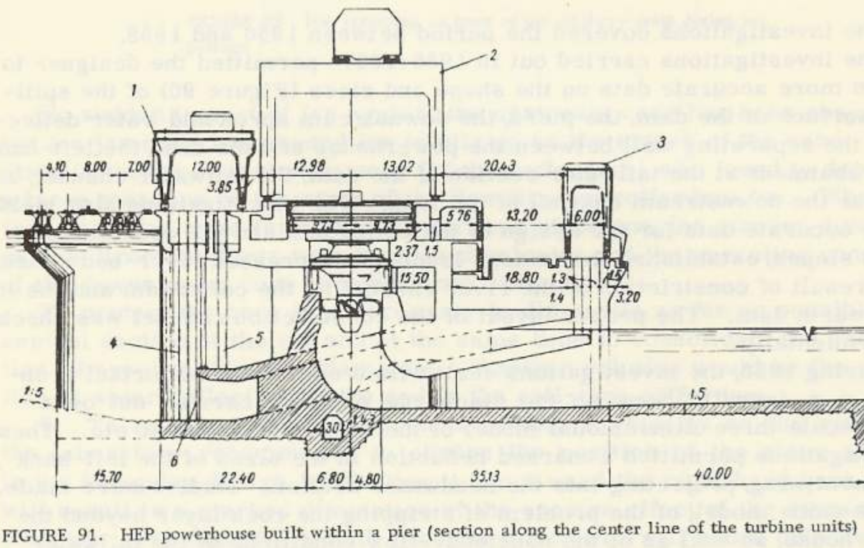


FIGURE 91. HEP powerhouse built within a pier (section along the center line of the turbine units)

(1) - 200-ton gantry crane; (2) - 500-ton gantry crane; (3) - 125-ton gantry crane; (4) - slot recess for the repair gates and the crane; (5) - slot recess for the working gates; (6) - coarse trash rack of the water-outlet structure.

These investigations established a series of hydraulic characteristics for the design, such as the discharge capacity of the development, the general pattern of streamflow downstream from the structures, distribution of the streamflow downstream from the apron and the water deflector, and the navigational conditions in the tailwater section.

The pier type of HEP as designed for the Dneprodzerzhinsk hydro development is inferior in its hydraulic characteristics to the first (basic) variant. This refers particularly to the navigational conditions in the tailwater section. It does not, however, exclude the adoption of the pier type for other conditions.

All the results were submitted to the respective design institute.

HYDRAULIC LABORATORY INVESTIGATIONS ON THE PROTECTIVE STRUCTURES OF THE FOREBAY OF THE DNEPRODZERZHINSK HEP

Responsible for Research: I. Ya. Popov, Senior Engineer

For the protection of the harbor located within the forebay of the Dneprodzerzhinsk HEP, the Ukrainian Branch of the Gidroenergoproekt developed two layout variants for the protective structures.

In the first variant the harbor is protected by a breakwater dike whose head section is parallel to the dam and is connected with the first section, normal to the dam, by a curvilinear intermediate section having a radius of 600 m.

The second variant differs from the first in that the angle between the center lines of the head and the first sections of the dike is 120° and not 90°.

A special unit, the so-called ondoscope (a small-scale harbor model) (see Figure 92), was used to test and compare the two variants. On the basis of the test results the laboratory concluded that:

1. A dike whose head section is parallel to the center line of the dam ensures more favorable wave conditions in the harbor than a dike built as in the second variant.

2. To eliminate any possibility of formation of reflected waves which would create unfavorable conditions of approach to the forebay, it is necessary to give the bank section AB (Figure 93) adjacent to the gate site, a gentle slope with an elevation of $m = 3.5$ to 4.0.

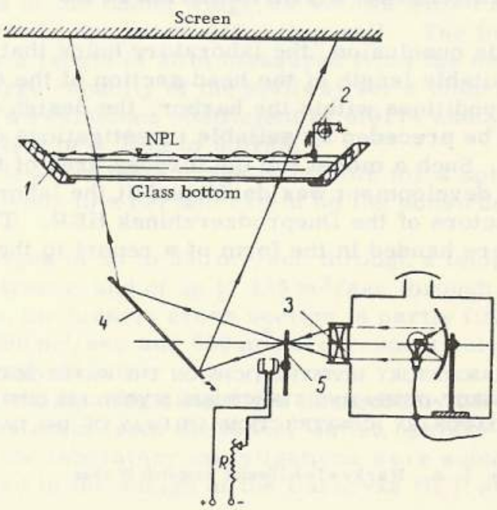


FIGURE 92. Schematic diagram of the ondoscope

1 - tank; 2 - wave generator; 3 - condenser lenses; 4 - mirror; 5 - stroboscope.

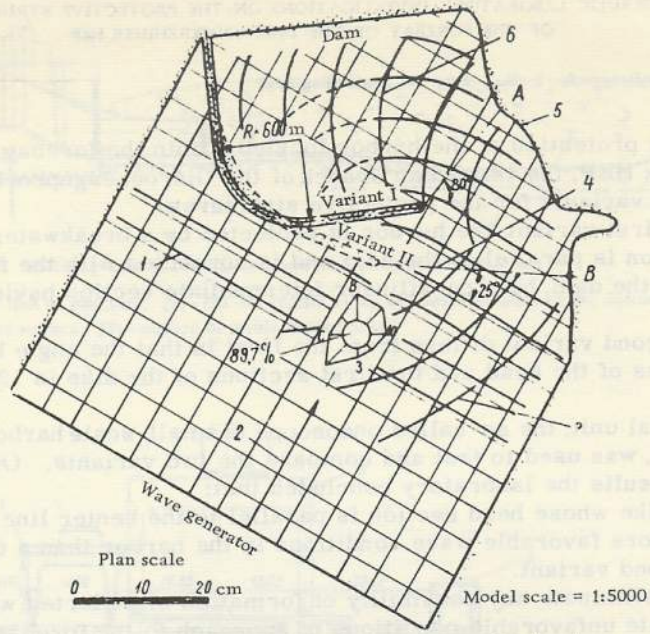


FIGURE 93. Bank section with gentle slope to prevent reflection of waves

1 - front of incoming wave; 2 - direction of waves; 3 - waves reflected from the breakwater; 4 - gate line according to design variant II; 5 - contour of storage reservoir; 6 - waves reflected from the bank.

Apart from this conclusion, the laboratory holds that, in order to determine the most suitable length of the head section of the dike and to obtain favorable wave conditions within the harbor, the design of the protective structure should be preceded by suitable investigations on a large-scale hydraulic model. Such a model for the forebay area of the headwater section of the hydro development was developed at the laboratory and was handed to the constructors of the Dneprodzerzhinsk HEP. The results of the investigations were handed in the form of a report to the Ukrgideg.

HYDRAULIC LABORATORY INVESTIGATIONS ON THE WATER-DISCHARGE TUNNEL AND THE ENERGY-DISSIPATING STRUCTURES BEYOND ITS OUTLET AS WELL AS ON THE TEMPORARY (CONSTRUCTION) SPILLWAY OF THE DAKHOVKA HEP

Responsible for Research: L. A. Narkevich, Senior Research Worker

The investigations on the water-discharge and spillway structures of the Dakhovka HEP were carried out in the years 1957 and 1958 and were a continuation of the studies started in 1955. These investigations dealt with the design of the discharge tunnel and the temporary (construction) spillway, and with the selection of proper energy-dissipating structures located downstream from the outlet of the tunnel.

As far as the construction spillway is concerned, the object of the present study has been to check the spillway design developed by the Mosgidep on the basis of laboratory research (1955-1956).

In connection with the reduction of the tunnel length to 691 m and changes in its route and entrance edge, it became necessary to make more detailed studies of the discharge capacity of the outlet structure, the hydraulic regime of stream flow, the passage of logs, and the operation of energy dissipators located downstream from the outlet.

The investigations were carried out on two three-dimensional models. The model of the discharge tunnel was made of timber and the river-channel of concrete; a pit, filled with sand during erosion tests, was used to test the erosion of the outlet-channel bottom downstream from the tunnel outlet.

The model for the tunnel of the temporary spillway was made of plexiglass, the model of the entrance head and of the energy dissipators, of timber, while the model of the storage reservoir and of the river channel below the tunnel outlet were made of concrete. The tests on the tunnel model were carried out at a flow rate of $110\,m^3/sec$ and at different levels of the tailwater section. The discharge capacity of the temporary spillway was checked against the maximum rate of $765\,m^3/sec$, while the log passage conditions were checked for discharges ranging from $150\,m^3/sec$ to $765\,m^3/sec$.

The above investigations permit the following conclusions to be drawn:

1. In order to ensure optimum spreading of the stream and to reduce the specific discharge downstream from the tunnel outlet, the width of the outlet at the tunnel wings has to be increased to 12 m. Furthermore, for an effective protection against erosion of the discharge-channel bottom, it is advisable to construct energy dissipators provided with a water-deflecting wall near the ends of the tunnel wings, as well as with a series of baffles at a distance of 9 m from the water-deflecting wall. The intermediate protection structure has a length of 30 m measured from the deflecting wall.

2. The discharge capacity of the spillway for a tunnel length of 691 m is reached taking a resistance coefficient of 0.0174 and a corresponding roughness coefficient of 0.0163 to 0.0165.

3. The water level in the storage reservoir for a spillway of a capacity of $765\,m^3/sec$ depends to a certain extent on the construction of the tunnel entrance.

4. For discharges of up to $350\,m^3/sec$ through a temporary spillway with 10 m wide entrance, and of up to $435\,m^3/sec$ through spillways with a 9 m wide entrance, the tunnels cross section is partly filled, while at discharges of over $550\,m^3/sec$ and $580\,m^3/sec$, respectively, the tunnel cross section is completely filled.

5. The energy-dissipating structures designed by the laboratory in 1956 and installed downstream from the tunnel outlet, operate satisfactorily.

The results of the laboratory investigations were submitted to the Mosgidep to be used in the design of the Dakhovka HEP structures.

HYDRAULIC LABORATORY INVESTIGATIONS OF THE SLUICEWAYS AND OVERFLOW SPILLWAYS OF THE UPPER TULOMA HEP

Responsible for Research: N. B. Isachenko, Candidate of Technical Sciences, Senior Research Worker

The aim of these investigations was to work out recommendations for the operation of sluiceways and overflow spillways at the Upper Tuloma HEP.

The research was carried out on two models:

1) a 1:50 scale model for the stretch of the river containing the temporary sluiceways; and

2) a 1:40 scale model for the overflow spillway, consisting of a timber chute with a small portion of the adjacent tailwater section.

The temporary spillway was designed as an intake tower provided with bottom and overflow openings, and connected to conduits through which the water was discharged into the tailwater section. During the initial period of operation, the water was discharged through the conduits which were later closed, while in the second period the water was discharged through the overflow outlets.

While the water is discharged through the conduits, provision should be made for the passage of logs.

On the basis of these investigations, the laboratory recommends the following:

1) to adopt the design providing for a temporary sluiceway consisting of three sections instead of four, which ensures savings in concrete of 6300 m^3, and in excavated rock of about 132,000 m^3;

2) to replace the stilling pool which, according to the original design, was to be built downstream from the floor conduits, by special energy dissipators with roughness ribs which ensure the dissipation of energy at various stages of operation;

3) to dispense with the erection of intermediate piers at the spillway of the head structures in order to improve flow conditions in the spillway chute, and to discharge the stream through one bay (with a span of 35 m) closed by a lenticular type of segment (Taintor) gates 35 m long and 7 m high.

4) to provide a funnel-type pit at the end section of the spillway.

IMPROVEMENTS IN THE OPERATION OF THE SETTLING BASIN AT THE ORDZHONIKIDZE HEP

Responsible for Research: P. I. Sokerin, Senior Research Worker

Poor operation of the settling basin of the Ordzhonikidze HEP is the result of imperfections in the design of the silt-flushing system. The small openings in the flushing system become easily clogged with trash and larger particles of sediment and, even before the end of the flood, the deposition of silt interferes with the proper functioning of the settling basin. The basin is cleaned mechanically throughout the winter.

The investigations showed that, in order to improve the operation of the settling-basin, it is essential to collect also the larger particles of sediment and to remove them by means of additional silt-removing systems provided at the settling basin inlet.

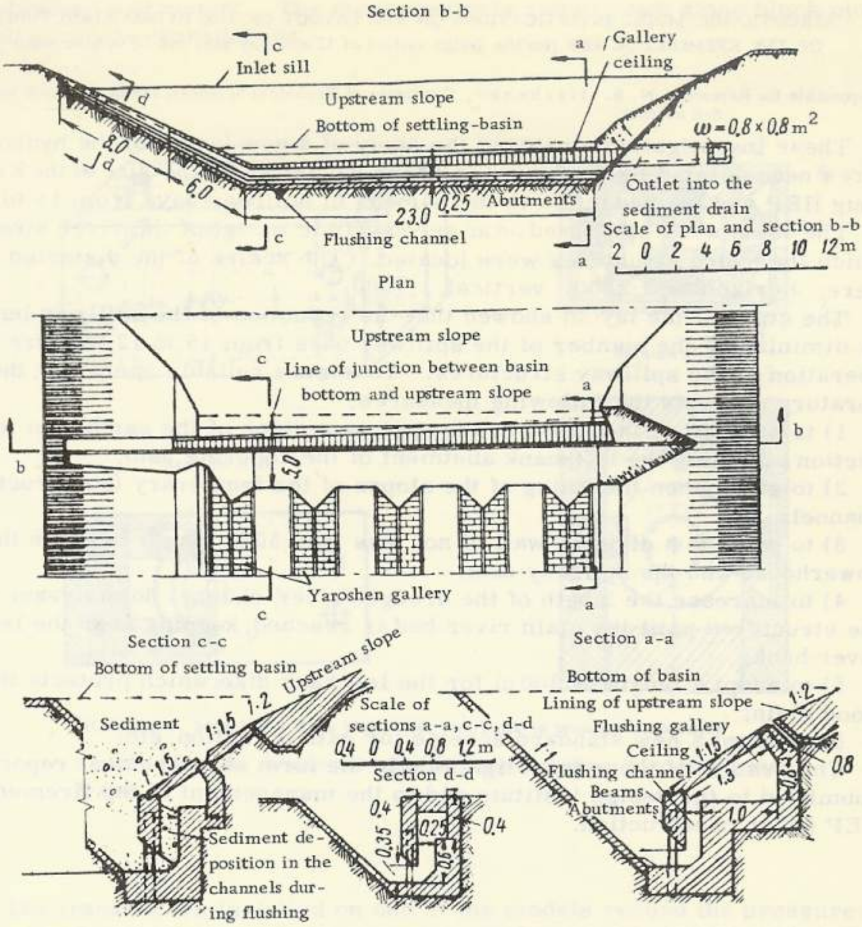


FIGURE 94. Flushing system for collection and removal of large-size sediment particles at the settling basin of the Ordzhonikidze HEP

Figure 94 shows the design suggested for such an additional flushing system. It consists of a flushing gallery of varying cross section, provided with a flushing channel. In this design sediments are not flushed simultaneously along the whole length of the gallery, but progressively, starting from its upper end. This ensures a maximum silt content of the flushing water, throughout the whole flushing period. Due to the relatively reduced cross-sectional area of the gallery, the discharge of flushing water will not exceed 2.3 m^3/sec. The large width of the continuously flushed channel will prevent the clogging of the gallery.

The advantages of this new flushing system led to its being recommended as a means for improving the Ordzhonikidze HEP settling basin.

AERODYNAMIC MODEL INVESTIGATIONS ON THE LAYOUT OF THE HYDRO STRUCTURES OF THE KREMENCHUG HEP (for the design variant of 12 spillway bays and 12 turbine units)

Responsible for Research: N. B. Isachenko, Candidate of Technical Sciences, Senior Research Worker

These investigations involved the study of a new layout of the hydro structures necessitated by the increase in the capacity and the size of the Kremenchug HEP and the reduction in the number of spillway bays from 15 to 12.

The tests were conducted on an aerodynamic model of the river stretch in which the hydro structures were located. The scales of the distorted model were: horizontal, 1:2000; vertical, 1:1000.

The study of the layout showed that the reduction of the spillway length by diminishing the number of the spillway bays from 15 to 12 impairs the operation of the spillway structures. To ensure reliable operation, the laboratory suggests the following measures:

1) to strengthen the lining of the upstream slope of the earth dam in the section adjoining the left-bank abutment of the concrete dam;

2) to strengthen the lining of the slopes of the temporary (construction) channel;

3) to provide a dividing wall of not less than 50 m length between the powerhouse and the spillway dam;

4) to increase the length of the dredged river-channel downstream from the structures until the main river bed is reached, keeping near the left river bank;

5) to adopt a length of 300 m for the left bank dike which protects the flood plain;

6) to adopt a new standard scheme for gate operation, etc.

The results of these investigations in the form of a technical report were submitted to the design institute and to the management of the Kremenchug HEP under construction.

DESIGN OF INSTRUMENTS FOR SCIENTIFIC RESEARCH

Responsible for Research: E. A. Genina, Senior Engineer

The main object of the research consisted in developing designs for two new types of instruments: a) pressure transducers for measuring the pressure on the face of rock models; and b) wave recorders for field conditions.

The work also included supervision of the manufacture of pressure transducers intended for use under field conditions. Several types of transducers used for measuring the pressure on the faces of stone-block models were designed, constructed, and tested. In view of the small sizes of the model and the necessity of installing four transducers, a special type of inductive nondifferentiating pressure transducer was designed.

Figure 95 shows the general view of the model together with the built-in transducers. The pressure to be measured is applied through an opening to a diaphragm which, when deflected, causes a change in the clearance between the diaphragm and the core, thus leading to a variation in intensity of the current flowing through the coil; this variation is recorded by a

measuring instrument. The measurements require two stone block models with pressure transducers.

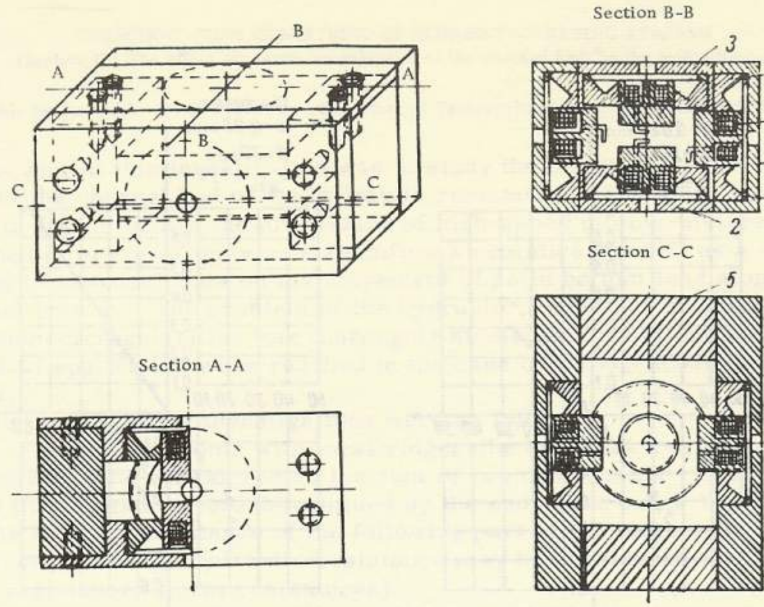


FIGURE 95. Model of a stone block with four built-in pressure transducers

1 and 2 - side plates; 3 - core; 4 - coils; 5 - connecting piece; 6 - transducer body; 7 - ring.

The transducers installed on one of the models record the pressure; the other model, with transducers whose coils represent the fourth arm of a Wheatstone bridge, is mounted under the same temperature conditions as the first model, but the transducers do not pick up the pressure. The study involved testing a model with one transducer measuring pressure on one face. The rating characteristics of an experimental transducer showed its sensitivity to be satisfactory (Figure 96).

For the use of such transducers in laboratory practice it is necessary to prepare a stone-block model containing all the four pressure transducers, and to test the unit under field conditions.

The laboratory designed the construction of a field-wire-type wave recorder for measuring the parameters of wave changes at the slope of water-retaining structures. The sensing element of the transducer consists of two steel wires whose surface, except for the contact area, is coated with an insulating layer of BF-2 glue. The wires are located within a protective Dural tube. The laboratory also prepared the necessary working drawings. The wave-recording unit, developed on the basis of the laboratory-type wave recorders, was supplemented by a device compensating the signal for the loss caused by the initial sinking of the wave recorder into the slope. Certain changes in the electric wiring diagram (insertion of a transformer

between the wave-recorder circuit and rectifying block) permitted rating characteristics) to be obtained, having a rectilinear shape within the basic working range (Figure 97).

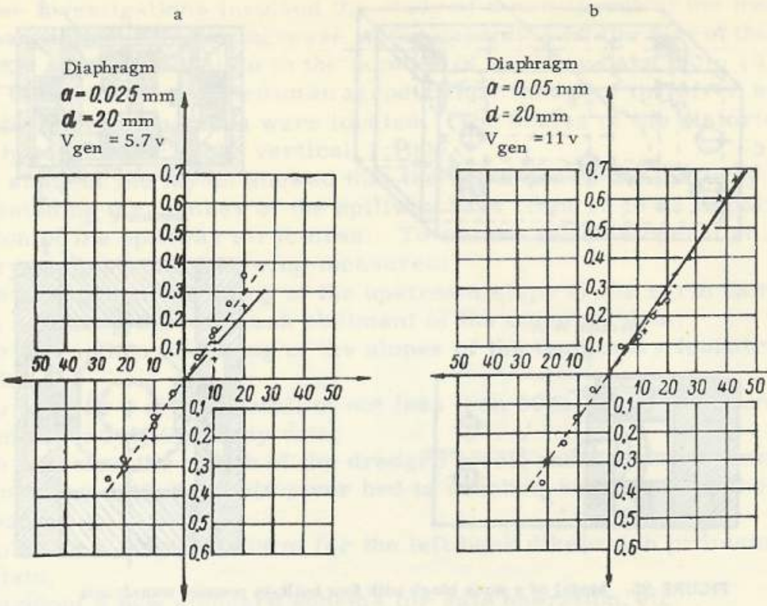


FIGURE 96. Rating graphs for transducers mounted on stone-block models

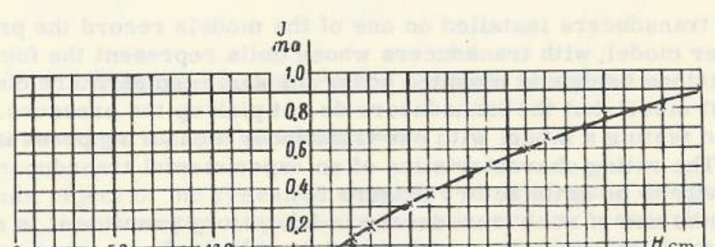


FIGURE 97. Calibrating graphs for a field wave recorder

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR DAMS AND HYDRO DEVELOPMENTS

Head: D. I. Kumin, Doctor of Technical Sciences

TURBULENT-FLOW CONDITIONS OF SEDIMENT-CARRYING STREAMS
(Section II: The effect of macro-irregularities of the channel beds on the hydraulic resistance)

Responsible for Research: V. S. Knoroz, Candidate of Technical Sciences, Senior Research Worker

The aim of this investigation was to study the effect of macro-irregularities of the channel bed on its hydraulic resistance. The study involved tests in a wind tunnel, the processing of high-speed motion-picture data obtained in previous tests on streamflow kinematics, as well as a critical review of research data on the movement of solid bottom load along the channel bottom. The problem of the hydraulic resistance of channel beds with macroirregularities was thoroughly studied, and solutions suitable for practical application were reached in the case of homogeneous sandy sediments.

The results of the investigations were as follows:

1. For river bottoms with local ridges, the hydraulic resistance of river channels may be considered as the sum of two components: local resistances (form resistances) determined by the eddy zone below the ridge crest, and the friction resistance of the following part of the ridge: for bottoms with local reefs, the hydraulic resistance may be considered as the sum of local resistances (form resistances).

2. For constant values of the qualitative and quantitative characteristics of the shape of an uneven sandy river bottom, the character of the variations in the coefficient of hydraulic resistance λ_r is similar to that of the variations found by Nikuradze and Zegzhda for pipes and open prismatic rectilinear channels with rough granular walls.

3. The coefficient of hydraulic resistance (λ_r) of uneven channel beds may be determined (within the limits of the "self-modeling" zone of resistance) from the formula

$$\lambda_r = \left(0.25 - 0.05\log\frac{h_r}{k}\right)\frac{h_r}{L_r}\left(\frac{h_r}{R}\right)^{0.25} + \alpha\lambda_0\left(1 - 10\frac{h_r}{L_r}\right), \tag{1}$$

where $\alpha = 1 - \left(\frac{h_r}{R}\right)^2$,

h_r, L_r and R = respectively the height of ridge or reef, their length, and the hydraulic radius of the stream, measured in the region of the ridge crest;

k and λ = the effective roughness of the stream and the coefficient of hydraulic friction of the stream in a channel bed with granular roughness (k) for stream and channel characteristics corresponding to those at the region of the ridge crest.

Remark. For a reef-shaped channel bottom, the coefficient λ_r is given by the first term of formula (1).

4. The boundary of the "self-modeling" and transitional region of resistance of a macroirregular channel bed may be found by

$$\mathrm{Re}_{kv} \geqslant \frac{63}{\sqrt{\lambda_r}} \cdot \frac{R}{k_e}, \quad (2)$$

which is similar to the formula developed by Zegzhda.

The equivalent roughness k_e (in the above formula) may be determined from formula

$$\log \frac{R}{k_e} = 0.25 \left[\frac{1}{\sqrt{\lambda_r}} - 4.25\right], \quad (3)$$

where λ_r is found from (1).

5. The resistances in the transitional region may be found from the Zegzhda formulas substituting for k (effective roughness), the equivalent roughness k_e determined from (3).

6. The investigations failed to establish any direct effect of solid-load discharge of the river on the hydraulic-resistance coefficient λ_r. The discharge of solid load has apparently an indirect effect on λ_r, inasmuch as the discharge influences the nature and dimensions of the bottom formations of the eroded bed.

The relationships obtained may be used in all relevant cases.

TESTING UNDER FIELD CONDITIONS THE RESULTS OF LABORATORY INVESTIGATIONS OF HYDRAULIC PROCESSES AT HYDRO DEVELOPMENTS

(Section III: Field investigations at the Dubossary HEP)

Responsible for Research: A. S. Abelev, Candidate of Technical Sciences, Senior Research Worker

The object of the present study was to test the results of laboratory investigations of the hydraulic regime at hydro developments, and to check operating conditions of hydro structures. The work was carried out by the VNIIG jointly with the research department of the Ukrgidep,which was in charge of the field investigations. The program for these field investigations was worked out by the VNIIG who was also responsible for the comparison of laboratory and field results, the assessment of the state of the river channel in the headwater and tailwater zones, the development of the practical measures required for the reliable and safe operation of the structures, and, finally, for the drawing up of general conclusions.

The investigations lead to the establishment of a systematic control of the channel conditions at the headwater and tailwater sections of the Dubossary HEP. Ample data were also obtained from field investigations affording a better understanding of the nature and extent of channel-erosion processes, distribution of surface flow velocities, distribution of flow velocities over the stream depth, as well as a clearer insight into the changes of the geological structure of the river channel as a result of the morphological changes due to scouring processes. Operating conditions at the hydro development were subject to detailed investigations, and the results recorded and systematized.

The results of the field investigations cannot be directly compared with the laboratory data, since they were conducted under different limiting conditions in respect of the spillway discharge and the channel conditions at the tailwater section. The results of the model tests can be compared with actual field conditions only by conducting new laboratory tests on models simulating exactly the boundary conditions under which the field investigations were carried out. The results of the field investigations permit conclusions to be drawn on the channel conditions at the headwater and the tailwater sections, and they afford a basis for practical recommendations for ensuring the reliable operation of the structures.

No marked changes were noticed in the channel structure on the headwater side of the Dubossary dam. The tailwater section exhibits unsteady local channel scouring, the maximum depth of scour being observed near the fourth and fifth spillway bay through which most flood discharges are diverted.

Deep local scouring in the tailwater section of the Dubossary HEP is the result of an incorrect discharge regime during construction and operation, which did not follow the precise instructions given by the laboratory and the design institute, and led to concentrated discharges and flashy streamflow.

The flexible downstream apron which was designed (disregarding the advice of the laboratory) as a series of hinged reinforced-concrete slabs placed above the rock fill, suffered considerable damage and is in a poor condition. This type of apron complicates the repair of the underlying rock fill, which, in itself, is a reliable and easily repairable lining.

Despite the pressure of fairly deep local scour, the general condition of the channel in the tailwater section does not constitute a direct danger to the stability and the normal operation of the hydro structures.

However, as the channel erosion did not become stabilized, and the scour funnel is gradually approaching the apron, and in view of the unsatisfactory condition of the flexible apron and the underlying rock fill, the research staff, in its conclusion on the 1957 investigation, recommended the following:

1) to repair during 1958 the end section of the apron by adding riprap of 30 to 40 cm stones placed over the flexible apron. (This recommendation has not been carried out up to the present, and the work should be done without further delay);

2) to adhere strictly to the schedule for the operation of the dam gates; the spillways should be opened only in exceptional cases (when required by operational conditions);

3) to continue the systematic observation of the condition of the channel in the tailwater section of the hydro development.

TESTING UNDER FIELD CONDITIONS THE RESULTS OF LABORATORY INVESTIGATIONS OF HYDRAULIC PROCESSES AT HYDRO DEVELOPMENTS

(Section V: Field investigations at the Novosibirsk HEP)

Responsible for Research: A. A. Kruchinina, Candidate of Technical Sciences, Junior Research Worker

The aim of this study was to test, under field conditions, the results of laboratory research on the hydraulic regime at the hydro development, and

also to check the state of the river channel in the tailwater section after the passage of floodwater through the structures. The study was carried out by the VNIIG jointly with Novosibirskgesstroi which was in charge of all field investigations, while the VNIIG prepared the investigation program, systematized the results obtained, and evaluated the state of the river channel in the tailwater zone.

As a result of these investigations a clear picture was obtained of the nature and the extent of channel-bed erosion, the condition of the energy dissipators at the downstream apron, and the passage of ice and floodwater through the structures during the last years of construction. The results of the field investigations cannot, at the present stage, be compared with the laboratory data, since they were carried out under different limiting conditions of water discharge, and at different construction stages of the water outlet structures. Furthermore, in the laboratory tests corresponding to conditions during the construction period, the rock of the channel bed was assumed to be nonerodible, and this assumption proved to be incorrect.

The investigations showed localized, unstabilized erosion at the tailwater section of the Novosibirsk HEP overflow dam, the maximum depth of scour being located at the seventh spillway bay (counted from the powerhouse).

The deep local scouring in the tailwater section is the result of not having followed the recommendations of the laboratory regarding the discharge regime, and this resulted in a concentrated flow of high specific discharge at the right half of the overflow dam. The state of the river bed at the tailwater section doss not constitute a direct danger to the stability and the normal operation of the structures. However, in view of the fact that the channel erosion in this has not become stabilized, it is necessary: 1) strictly to adhere to the laboratory recommendations concerning gate-operating schedules; 2) to continue systematic observations on the state of the river bed in the tailwater section, increasing the range of observations to a length of 200 to 300 m of river bed; 3) to strengthen the channel consolidation downstream from the apron cut-off by applying an additional riprap layer of suitable stone size.

Observations on ice passage through the Novosibirsk hydro structures confirmed that the decisive factor is the thickness of the ice sheet.

The observed deterioration of the energy dissipators at the downstream apron of the dam was found to be the result of cavitation. It is therefore necessary to carry out periodic inspections of the energy dissipators and to repair them when necessary.

TECHNICAL, ECONOMIC, AND EXPERIMENTAL CONSIDERATIONS IN THE DESIGN OF STANDARD HYDRO UNITS WITH OPTIMUM SIZES OF THE WATER PASSAGES FOR RUN-OF-RIVER POWER PLANTS

Responsible for Research: V. A. Solnyshkov, Junior Research Worker

The search for new, more efficient shapes of draft tubes started as far back as 1952 and continued in 1958. In the previous years the laboratory developed two types of bench-tested draft tubes, having a diameter D_1 of 100 mm, together with a runner type PL-510 with a blade angle of $\varphi = +5°$.

The objective of the present study was to establish more exact power characteristics of the new draft tube designs for three types of wheels: PL-510, PL-587, and PL-661*); for a range of blade angles φ from $-10°$ to $+20°$. The method for evaluating the power characteristics of the draft tube was the same as in previous studies, i.e. by noting the difference between the efficiency of the turbine with and without the draft tube.

The results of testing the VNIIG-type draft tubes are included in the table of over-all characteristics of the turbines.

The final evaluation of the power characteristics of the new types of draft tubes compared with the LMZ-No. 4 draft tube can be obtained from the analysis of the over-all characteristics of the PL-587-10, PL-510-10, and PL-661-25 turbines. The Coriolis coefficients, calculated from the exit-velocity distribution permit evaluation of exit losses in the VNIIG and LMZ-type draft tubes (VNIIG-I $\alpha_{exit} = 1.09$; VNIIG-II $\alpha_{exit} = 1.32$; LMZ No. 4-a $\alpha_{exit} = 1.38$).

Tests of VNIIG draft tubes showed the efficiency of these types to vary with the type of runner and the operating conditions.

The table below shows how to select the turbine runner and operating conditions for maximum efficiency of draft tubes of the VNIIG-I, VNIIG-II, and LMZ No. 4-a types.

TABLE

Operating conditions \ Type of runner	PL-587	PL-510	PL-661
Optimum	Draft tube VNIIG-I, LMZ No. 4-a	Draft tube VNIIG-I	Draft tube VNIIG-II
High speed	Draft tube VNIIG-I, LMZ No. 4-a	Draft tube LMZ No. 4-a	Draft tube VNIIG-I and LMZ, No. 4-a

Thus, for run-of-river power plants with vertical turbine units, it is advisable to use LMZ No. 4-a draft tubes as well as the VNIIG I draft tube for PL-587 and PL-510 runners, and the VNIIG-II tube for PL-661 turbine runners.

* [According to Soviet designation PL stands for adjustable-blade wheels.]

NEW METHODS FOR THE DESIGN AND CALCULATION OF HYDRAULIC GATES, TAKING INTO ACCOUNT LOAD FLUCTUATIONS ARISING DURING THE STREAMFLOW AROUND THE GATES

Responsible for Research: A. S. Abelev, Candidate of Technical Sciences, Senior Research Worker

Fluctuations of hydrodynamic loads acting on the gates while the water stream is flowing around them cause vibrations both in the gate-supporting frames and in the adjoining concrete structure. These vibrations have a damaging effect on the gate structure and reduce its service life. Moreover, under conditions of saturated sandy foundation, the gate vibrations transmitted through the concrete structure seriously reduce its structural stability. Despite the importance of these problems they have so far not been given sufficient consideration.

The aim of the present study has therefore been to carry out a detailed investigation of gate vibrations and their causes, and also to find reliable methods for the elimination of these dangerous phenomena.

In view of the complexity of these phenomena, it is not possible to study them on models, and it was decided to divide the laboratory investigations into two parts: 1) experiments on the fluctuations of total hydrodynamic loads (i.e. of perturbance forces) under various conditions of gate operation; 2) theoretical studies (based on the theory of forced oscillations) which permit the determination of the basic magnitudes characteristic of gate vibrations.

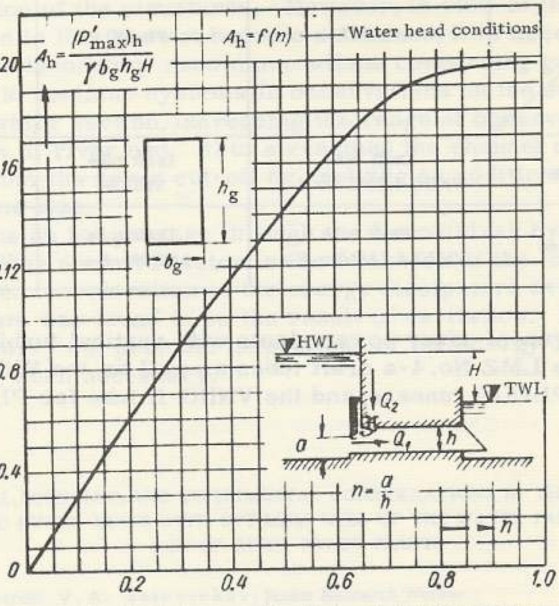


FIGURE 98. Graph for computing fluctuations of hydrodynamic loads acting on plain sliding gates in a horizontal direction

The study (a continuation of investigations started at the institute in 1955-1957) was mainly concerned with vibrations of total hydrodynamic loads (i.e. of perturbance forces) acting on butterfly-type and segment (Taintor) gates.

The vibrations were investigated by means of recording strain gages.

In 1958, work continued and was completed on the experimental investigation of fluctuations of total hydrodynamic loads on plain sliding gates, as well as of fluctuations of loads and moments acting on butterfly and segment gates.

After a first processing and analysis of the test data, general-purpose computation graphs were obtained for the determination of fluctuation amplitude and frequence of hydrodynamic loads acting on sliding, butterfly, and segment gates under various hydraulic-operating conditions.

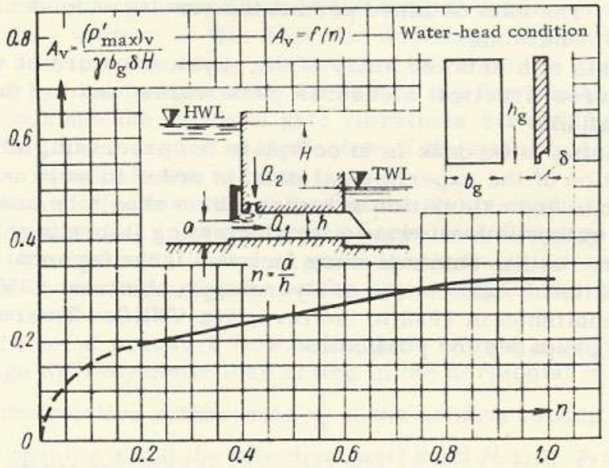


FIGURE 99. Graph for computing fluctuations of hydrodynamic loads acting on plain sliding gates in a vertical direction

Figures 98 and 99 show general-purpose graphs for computing the maximum fluctuation amplitude of the total hydrodynamic loads acting on plain sliding gates in the horizontal $(p'_{max})_h$ and vertical $(p'_{max})_v$ directions. The abscissa in these graphs show the relative opening of the gate $n = \frac{a}{h}$, while the ordinate gives the dimensionless number A, calculated from the following characteristic magnitudes

$$A = \frac{p'_{max}}{\gamma \omega H}, \qquad (1)$$

where $p'_{max} = p_{max} - p_{min}$ represents the maximum fluctuation amplitude of the total hydrodynamic load;

ω = area subjected to the fluctuating load; for loads acting in a horizontal direction, $\omega = b_g h_g$ (Figure 98) and for vertical loads, $\omega = b_g \delta$ (Figure 99);

γ = the volume weight of water;

H = effective head (i.e. difference of level between headwater and tailwater).

For a given gate layout with sizes h_g, b_g and δ, and effective head H, these graphs permit the determination of the horizontal $(P'_{max})_h$ and vertical $(P'_{max})_v$ loads for the pressure regime of the structures in any value of n between 0 and 1.0.

The investigations also yielded correction factors (regime coefficients) for the conversion of fluctuating loads upon transition from the pressure regime for which the graphs (Figures 98 and 99) were obtained, to other possible hydraulic regimes.

Other important investigations were carried out to establish the relationship between the fluctuation of total hydrodynamic load acting on the gate as a whole, and the fluctuations of the load acting on its various separate elements. These investigations permit a wider application of the results obtained earlier.

Another problem considered was that of simulating hydrodynamic load fluctuations. The data obtained permit the results of model tests to be applied to field conditions.

On the basis of a detailed study of the physical nature of vibrations of hydro structures, practical measures were worked out for the elimination of such vibrations.

What remains to be done is to complete the processing, analysis, and systematization of the experimental data, in order to work out practical recommendations and calculation schemes which should be used in the design of hydraulic gates, with allowance for fluctuating (vibrating) loads.

Part of the results obtained were included in the report to the VIII Congress of the International Association of Hydrology*, Montreal 1959) and also in two papers published in 1958 in the Izvestiya VNIIG. The rest of the material is being prepared for publication.

EXPERIMENTAL INVESTIGATION OF THE HYDRODYNAMIC FORCES OF A WATER STREAM ACTING ON SUBMERGED SLIDING GATES

Responsible for Research: A. S. Abelev, Candidate of Technical Sciences, Senior Research Worker

The purpose of the present work was to study the hydrodynamic action of a water stream on submerged sliding gates. Tests were carried out on gates located in shafts and provided with two-sided lining and a downstream seal.

The study program included:

1) investigation of fluctuations of hydrodynamic pressure in the area between the bottom edge of the gate and the upstream face of the spillway, in order to establish the relationship between the pressure fluctuations in this area and the fluctuations of the total hydrodynamic pressure acting on the gate;

2) investigation of average hydrodynamic loads acting on the gate in both horizontal and vertical directions, at different gate openings, at different water heads and discharges, different headwater and tailwater levels, etc.

* [This is apparently the meaning of the Russian abbreviation "MAGI" which may stand for "Mezhdunarodnaya Assotsiatsiya Gidrologov."]

The study started in 1957 and was completed in 1958.

Investigations on the first point of the program were conducted on an oscillating gate model to which six strain gages were fastened. The pressure fluctuations in the area between the downstream bottom face of the gate and the upstream side of the water passage (water flowpath) conduit channel were investigated by means of strain gages fastened to the upstream face of the water channel.

The averaged hydrodynamic forces acting on the gate in a horizontal direction, were measured by means of strain gages, and the vertical forces, by means of a special weighing arrangement.

These investigations confirmed the author's earlier assumption regarding the close connection between the pressure fluctuations in the area between the bottom edge of the gate and the upstream face of the conduit, and the fluctuations in the total hydrodynamic load acting on the gate. The admission of air into this zone of positive pressure in the conduit has, of course, no influence on the fluctuation amplitude of the total hydrodynamic load. If, however, there is a vacuum in this zone, air admission into the conduit will have a marked and, in some cases, an adverse effect on the fluctuation amplitude of the total hydrodynamic load. To reduce the fluctuation of the total hydrodynamic load and the extent of gate vibrations, air may be fed directly into this zone, but this measure is ineffective if the quantity of air supplied is insufficient to eliminate the vacuum.

Under conditions of pressureless flow in the water conduit, the maximum value of the hydrodynamic load acting in a horizontal direction may be determined from the hydrostatic law of pressure distribution, assuming the line of excess piezometric pressure on the upper edge of the gate to coincide with the headwater level.

Under conditions of pressure flow in conduits without supply of air to the gate, the average hydrodynamic load acting in the horizontal direction does not depend on the relative submersion $\frac{l}{h}$ of the conduit ceiling, but on the degree of gate opening n and the effective (net) head H, i. e. $p_2 = f(n, H)$. Under these conditions p_2 should be calculated from the formula $p_2 = 1.25\,\gamma H b_g h_g$, where H = difference between headwater and tailwater levels; b_g = span of gate subjected to the load; h_g= effective height of gate.

It was found that, in calculating the average hydrodynamic load acting in a vertical direction, the load exerted in the tailwater section on the bottom edge of the gate is not taken into account, the computed vertical load will, in most cases, be too high. The study presents formulas and graphs for the calculation of the average vertical hydrodynamic forces acting on submerged sliding gates. However, as the tests were carried out on small-scale models and the numerical values obtained have a low absolute magnitude, the data on the vertical forces must be considered as being only tentative. More exact results may be expected with the use of a larger-scale model and the determination of the exact piezometric pattern along the gate edges.

HYDRAULIC LABORATORY INVESTIGATIONS OF THE KRASNOYARSK HYDRO DEVELOPMENT

Responsible for Research: A. G. Solov'eva, Candidate of Technical Sciences, Senior Research Worker

The investigations carried out during 1955 and 1956 at the VNIIG laboratory of dams and hydro developments established the advantage of building at the spillway face of the Krasnoyarsk HEP dam, a high, smooth, upturned bucket for diverting the spilling water from the dam to a distance sufficient to prevent damage by scour.

Optimum values were found for the basic parameters of such a bucket, and the flow-junction conditions between the headwater and the tailwater sections of the dam were investigated.

The profile of the spillway dam recommended by the laboratory was adopted in the final design.

The Gidroenergoproekt institute examined the preliminary design of the hydro development and found insufficient basis for the choice of the type of streamflow junction. They advised carrying out further hydraulic investigations of the energy-dissipation system.

The institute also recommended that research be continued on upturned high-edge buckets with and without chute blocks and that new research be carried out on dams with low-edge upturned buckets of various designs, as well as on conventional buckets without upturned edge (stilling pools).

During 1957 and 1958 the investigations centered mainly on energy-dissipation problems.

Apart from dealing with the choice of the type of streamflow junction, the laboratory carried out during this period a detailed study on streamflow junctions and on erosion downstream from the dam, which was provided with a smooth upturned bucket, as well as on the navigation conditions in the tailwater area; methods were developed to ensure normal operating conditions at the HEP for different elevations of the dam crest and normal pondage level.

Other studies dealt with the water and ice discharge over the dam crest (for a buttress dam) and the determination of the pressure on the buttresses which causes the failure of one section of the dam. The main investigations were conducted on a three-dimensional, 1:170 scale model of the hydro development; for certain specific problems, use was also made of models made to a 1:80; 1:100; and 1:150 scale.

Model tests were carried out on dams with high-edge upturned buckets and, alternatively, with chute blocks, on dams with low-edge buckets of various designs, or with stilling pools; the purpose of these tests was to select the most suitable type of streamflow junction between the headwater and the tailwater section of the dam.

In deciding on the construction of energy-dissipating chute blocks, the possibility was considered of dividing the stream over the spillway face and the edge of the bucket into a number of nappes, the separate nappes flaring out during their free fall through the air, and as far as possible, colliding against each other. As a result, some of the energy is dissipated. At the same time the air entrainment into the nappe increases, and this widens its cross section and lessens the effect of the impact of the nappe on the river bottom.

The efficiency of these energy dissipators was judged by comparing the extent of the scouring action with that found in the case of a bucket with a

plain (nonupturned) edge, as well as by the distance of the area of maximum scour from the bucket edge.

The investigations covered 25 design variants of energy dissipators with high-edge buckets, and 2 variants with low-edge buckets.

Detailed studies also dealt with dams having plain buckets of different design and depth, as well as dams with stilling pools.

For each of these variants particular attention was paid to the flow regime of the stream, to the navigation conditions in the tailwater section, scouring downstream from the dam, and also to general operating conditions.

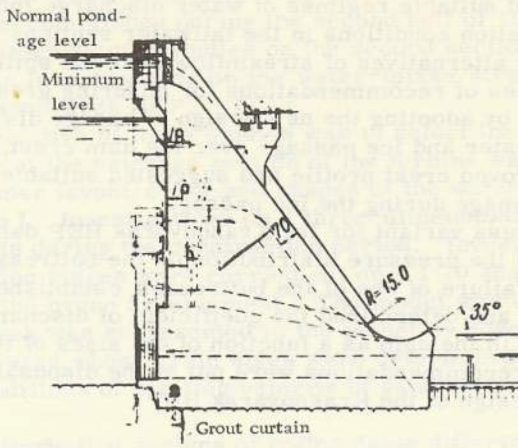


FIGURE 100. Section of dam with a short high-edge upturned bucket

These investigations established the main shortcomings and advantages of the different types of streamflow junction, and permitted the best design variant to be selected for the specific conditions at the Krasnoyarsk HEP.

The laboratory came to the conclusion that, under conditions prevailing at the Krasnoyarsk HEP, i.e. for a dam higher than 100 m on a massive-rock foundation, the most suitable type of water-energy dissipation appears to be a high-edge upturned bucket; the laboratory recommended not to alter the proposed design of the dam profiles with a bucket of $R = 15$ m radius of curvature, and an angle of jet exit $\alpha = 35°$ (Figure 100). With such a bucket, the jet is thrown a distance of 140 - 145 m from the structure, and the erosion starts only at a distance of 90 to 100 m from the dam; the maximum depth of scour occurs at a distance of 150 to 160 m from the structure, and reaches about 45 m* assuming the rock foundation as consisting of separate blocks of 1 to 1.7 m in size).

The laboratory came to the conclusion that the provision of chute blocks with this type of bucket is not to be recommended. On the one hand, they reduce the depth of scour at normal discharges by about 12%, but, on the other hand, the distance of the scour zone from the dam is lessened. Moreover, at smaller discharges, the effectiveness of the chute blocks decreases considerably.

* [Apparently misprint for 4.5 m.]

Such blocks also complicate operations since, in this case, the smaller degree of streamflow widening over the crest, and the danger of cavitational erosion of the blocks, require strict adherence to a definite sequence in the opening of the dam gates (opening firstly those bays with deflectors) in order to avoid oblique flow around the chute blocks, etc.

Apart from solutions to the problem of choosing the type of streamflow junction, these investigations also solved the problems of optimum length of the dividing pier between the powerhouse and the dam, of the length of the wall between the dam and the strip lift of the location of this wall with respect to the spillway bays, and of the elevation of the foundation; the investigations also established suitable regimes of water discharge intended to ensure favorable navigation conditions in the tailwater sections.

After studying other alternatives of streamflow over the spillway, the laboratory made a series of recommendations for ensuring greater reliability of the structures by adopting the new design of energy dissipators.

From the study of water and ice passage over the dam crest, the laboratory developed an improved crest profile and suggested suitable crest elevations for preventing damage during the ice passage.

In studying the buttress variant for the Krasnoyarsk HEP dam, the laboratory carefully analyzed the pressure distribution on the buttress surface for the case of a possible failure of one of the buttresses, established the factors affecting this pressure and determined the coefficient of discharge through the emergency opening in the dam as a function of the sizes of the opening.

All the results and recommendations were put at the disposal of the design institute for the design of the Krasnoyarsk HEP.

HYDRAULIC LABORATORY INVESTIGATION OF THE KRASNOYARSK HYDRO DEVELOPMENT

Responsible for Research: For Section I - A. G. Solov'eva, Candidate of Technical Sciences, Senior Research Worker
For Section II - G. A. Yuditskii, Candidate of Technical Sciences, Junior Research Worker
L. G. Moskvina, Senior Engineer

The research carried out at the laboratory for dams and hydro development of the VNIIG between 1955 and 1958 confirmed the advantage of erecting at the Krasnoyarsk dam a high-edge upturned bucket which deflects the water nappe flowing over the dam face, to a distance which is sufficient to eliminate scour at the dam foundation.

The optimum design parameters for such dam profiles were established, the conditions of stream junction between the headwater and tailwater were studied, and recommendations were made for the operation of the dam gates (considering both the navigation conditions at the tailwater section and the normal operation of the HEP).

During the same period, the laboratory also carried out model tests of the various construction stages of the hydro development, determined the elevation and the layout of coffer dams for the first and second construction stage, the water discharge capacity of the temporary water outlets, and the conditions of energy dissipation of the spilling water nappe. Apart from

this, the laboratory worked out measures to ensure the safety of temporary structures and made suggestions for the design of various component elements of the hydro structures.

All these studies and suggestions were taken into consideration in the design of the Krasnoyarsk HEP.

In the course of the design, it became necessary to introduce certain essential modifications in the first draft of the project, and this involved repeated model tests of all construction stages.

The main purpose of the first part of these studies was to investigate the conditions of water discharge over the whole construction period. This task was accomplished during the second half of 1958. At the same time the laboratory started studies on the second section of the program, viz., "Hydraulic investigations on the water-intake structures for the turbines of the Krasnoyarsk HEP.

The objective of these studies was to select the optimum design, the size and shape of the entrance section of the turbine water intake, and to arrive at the proper layout, sizes, and shapes of the water intake screens.

Section I. Investigations on a three-dimensional model of the hydro developments during the construction period. Investigations of the various construction stages were carried out on a 1:70 scale three-dimensional model of the hydro development. The model was of the movable bed type: the bed rock was represented in the model by concrete, and the alluvial deposits by sand whose grain sizes were determined so as to ensure similarity of the conditions of starting velocity of sand between the model and the prototype.

Five alternative designs of coffer dams differing in layout, degree of river-bed constriction, and design of structural elements, were used in the study of water-discharge conditions during the various construction stages.

The study of these design variants started with the passage of small discharges through the noneroded river bed, the discharge being gradually brought to the full rated value.

During the whole period of passage of temporary water discharges, the laboratory studied the level of the free surface of the river along the whole stretch under investigation, the flow velocity distribution, and the deformations of the alluvial deposits; navigation conditions were also investigated and suitable measures for erosion protection of the coffer dam slopes were worked out.

In order to determine the best methods for erecting the coffer dams, the laboratory tested on special models two designs for the coffer dam of the first construction stage.

The construction of the cofferdams for the second stage was studied in the case of one design only.

These investigations, in their final stage, involved the study of water passage over the dam crest and through the sluiceways (bottom outlets) in the first dam tier.

The model tests permitted the detailed study of all methods of closing the crest openings, their discharge capacity, as well as the streamflow regime in the tailwater section.

It is assumed that water will discharge through the spillways of the first tier at a head of up to 60 m; it was therefore necessary to study and select the best ways for dissipating the energy of the water flowing through these openings.

While the model tests on the water discharge through the outlets of the first tier neared completion, the building organization asked the designer to study the possibility of modifying the methods of delivering concrete aggregates to the construction site.

Usually, these materials were brought to the site by railroad or motor lorry. The changeover to water transportation and the choice of a proper location for landing stages involved detailed model tests. Since the landing stage for unloading the aggregates had to be erected while construction of the coffer dam for the first stage was still going on, its location had to be chosen considering the possibility of its use throughout the whole period of concrete placing.

The location of a landing stage depends in a large measure on the flow velocity distribution and the state of the alluvial river bed in the tailwater section. The studies on the problem of location required the repetition of investigations of flood passage both in the first and the second stage of construction for medium and maximum flood discharges.

During the tests, distribution of flow velocities and of deformation of alluvial deposits were studied.

The results of these investigations made it possible to select the optimum design for the coffer dam of the first construction stage, to establish the elevation of the coffer dam crest, to work out measures for slope protection against erosion, and to study conditions of navigation and ice passage through the free, nonrestricted, right river arm.

As a result of these investigations it was also possible to determine the discharge capacity of the dam crest and the discharge outlets in the first tier; the crest elevation of the dam for the second construction stage; the conditions for closing the crest outlets, and the streamflow regime in the tailwater section.

A rational method for energy dissipation of the streamflow through the outlets of the first tier was developed, the nature of the deformations of alluvial deposits in the river bed was clarified, the navigation conditions were investigated, and a first draft for the location of the landing stage was prepared.

The results in the form of two preliminary technical reports were referred to the design institute for the project of the Krasnoyarsk HEP.

Section II. Hydraulic investigation of the water intake structure for the turbines of the Krasnoyarsk HEP. The over-all sizes of the water-intake opening limited the laboratory investigations to small-scale models. On the other hand, the size of the component elements of the intake screens and gates did not permit small-scale model tests, in view of the considerable scale distortions due to the deviation from the Reynolds similarity conditions. Moreover, any exact small-scale geometric simulation of such gates is difficult due to the small sizes of their components.

The problem was therefore studied in two stages.

The first stage consisted of tests, on a separate schematic model, of a series of screens differing in the size of the bars and coefficient of obturation (ε). All the screens tested consisted of vertical, rectangular bars, the ratio between the sides of the section being 1:8.

The coefficients of hydraulic resistance ζ obtained for such screens for a wide range of Reynolds numbers permitted choosing the dimensions of the screen elements for the water-intake model so that the coefficient of hydraulic resistance of the model be equal to that of the full-scale prototype intake.

Figure 101 shows the computation graph $\zeta = f(\varepsilon, Re_b)$ (b = distance between screen bars) obtained from these model tests.

The second stage consisted in studying the hydraulic regime of the water intake as a whole. The trash racks were dimensioned according to the test results of the first stage, and were mounted on a 1:80 scale model of the water intake.

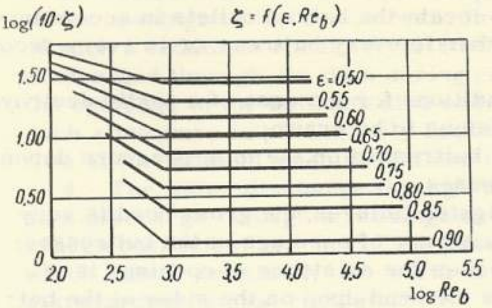


FIGURE 101. Graph for computation of the relationship $\zeta = f(\varepsilon, Re_b)$.

A comparison was made between two designs of screen frames for the water intake, and criteria for their selection were established. The component elements of the water intake were evaluated from the point of view of their influence on the general flow pattern, and the numerical values for the hydraulic-resistance coefficients of the water intake were calculated.

The design of the water intake was found to be satisfactory, the resistance coefficient ζ being 0.20.

Both design variants for the screen frames proved to be of equal value. The laboratory recommends a screen frame which is curvilinear in plan (design No. 1).

The results of these investigations, in the form of separate reports, were referred to the design institute responsible for the Krasnoyarsk HEP project.

HYDRAULIC LABORATORY INVESTIGATIONS OF SOME ADDITIONAL PROBLEMS CONCERNING THE PROJECT OF THE KRASNOYARSK HEP

Responsible for Research: A. A. Kruchinina, Candidate of Technical Sciences, Junior Research Worker

The present annotation refers to investigations carried out in 1958 by the Laboratory for Dams and Hydro Developments on problems concerning the erection of massive buttressed dams.

The investigations were conducted on a 1:175 scale model of the Krasnoyarsk HEP (see Figure 102).

The following two problems were analyzed.

1) the hydraulic streamflow regime through a partly destroyed dam;

2) water discharges through bottom outlets (sluiceways) during the construction period.

For the first problem three cases of dam failure were considered:

1) one block (i.e. one buttress) destroyed completely;

2) one buttress destroyed to half its height;

3) one buttress destroyed down to three quarters of its height;

The following values were determined:

1) lateral pressure on the walls adjacent to the destroyed buttress;

2) amount of water discharge through the destroyed block.

For the purpose of the investigations two designs of the dam were considered:

a) with openings in the buttresses and the downstream face of the dam;

b) without openings in the buttresses, but with openings in the downstream face.

The second problem involved the study of the discharge capacity of the sluiceways in the first tier [of the spillway] at a tailwater depth of 20 m.

The design institute proposed to locate the bottom outlets in accordance with the results of the research, either in every buttress, or in every second buttress.

Investigations of streamflow conditions for the case of a partly destroyed dam permitted the following conclusions to be drawn:

1. The effect of openings in the buttresses on the total pressure depends on the extent of failure of the buttresses.

a) for the smallest of the investigated failures, the gross head is very small and does not depend on the existence of openings in the buttresses;

b) for medium failures of buttresses the existence of openings is of great importance; without openings the head drop on the sides of the buttress adjacent to the destroyed block, attains 40 to 50 m, whereas openings reduce this difference to about 10 to 15 m;

c) for complete failure of the buttress, the existence of openings is also important; without them the head drop reaches values of up to 60 m, while with openings, this difference drops to 45 —50 m.

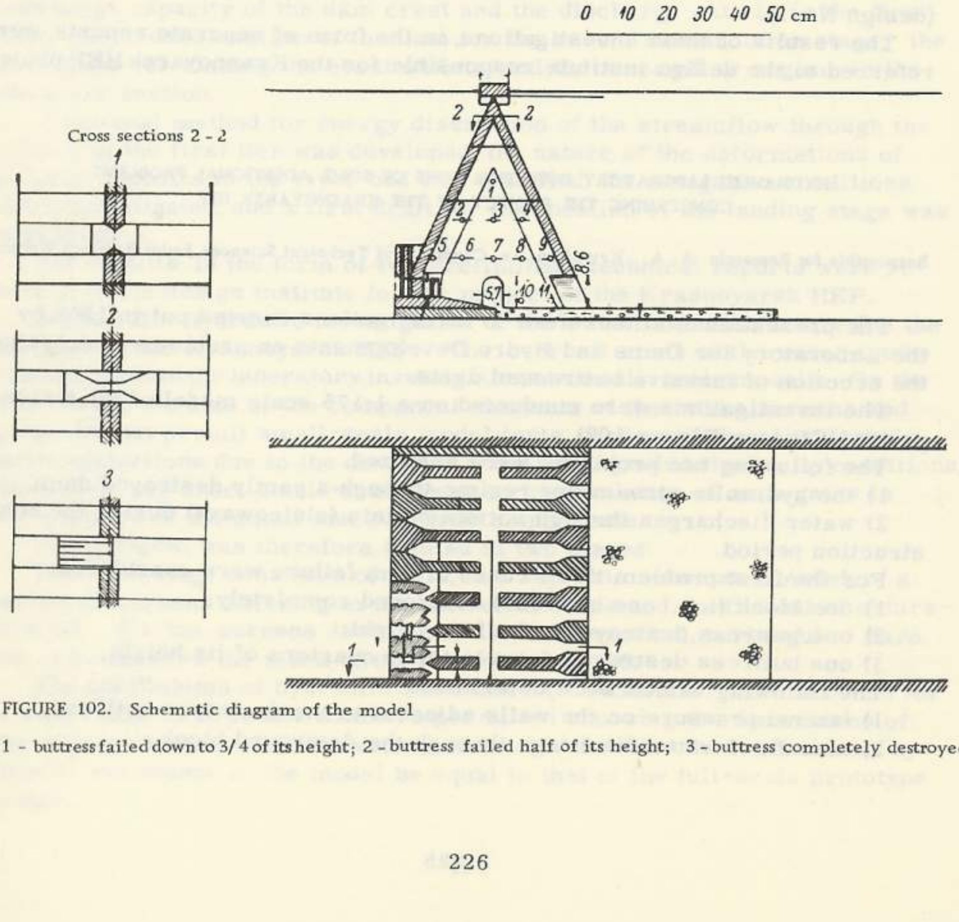


FIGURE 102. Schematic diagram of the model

1 - buttress failed down to 3/4 of its height; 2 - buttress failed half of its height; 3 - buttress completely destroyed.

2. In the design alternative with transverse openings a partial destruction of the dam buttress (block) is not dangerous with respect to the lateral pressure on the buttress adjacent to the damaged structure. On the other hand, complete destruction leads to extremely large (destructive) loads (i.e. lateral pressure of up to 45 m or even more).

3. Considerable lateral pressure resulting from complete destruction of the buttress can be reduced to any desired value by reducing the sizes of the openings in the downstream face of the dam; however, the possibility of such a reduction should be related to the conditions under which water is discharged through the dam during construction. A maximum increase of the openings in the buttress also causes the lateral pressure to diminish; such a measure is particularly important during the period of nonsteady flow at the moment when the failure of buttress occurs.

4. The water discharge through the opening in the buttress is calculated as a discharge through a rectangular overflow in a vertical wall, with a discharge coefficient $m = 0.43$; the head is the head above the datum line at which failure occurred; the width is the distance between the buttress heads, disregarding lateral restraint.

Maximum discharge Q at a normal pondage level (NPL) = 100 m is as follows:

for a completely destroyed buttress, 27,600 m^3/sec;
for a partially destroyed buttress, 10,000 m^3/sec;
for minimal destruction, 3300 m^3/sec.

5. In partly destroyed buttresses, the stream flowing through the break is divided into two parts: one branch flows through the gap between the buttresses and passes into the tailwater through the opening in the downstream face of the dam; the other branch spills over the remnant of the buttress as a powerful jet falling at a distance of 25 to 50 m away from the dam base.

At a NPL of 100 m the water discharge separates into two nearly equal branches whereas at a lower pondage level most of the water discharges through the gap between the buttresses.

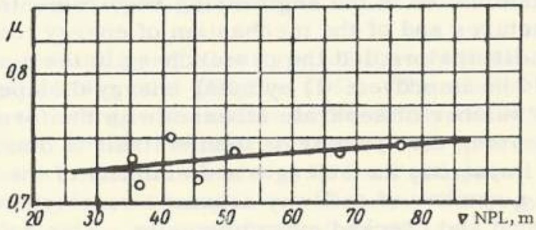


FIGURE 103. Graph illustrating the function $\mu = f(\nabla \mathrm{NPL})$.

6. With the passage of the water stream through the gap in the damaged buttress, the dam base erodes both within the dam structure proper and in the area of the tailwater section adjoining the dam base. With a completely damaged buttress the danger of erosion is due to large specific water

discharges (more than 1600 m^{2*}/sec); with a partly damaged buttress, the specific water discharge is markedly lower (70 m^{2*}/sec for average damage, and 30 m^{2*}/sec for minimum damage); however, in this case, the deciding factor is the height of the falling jet (30 and 56 m, respectively). To avoid scouring of the dam base, it has to be protected by a thick concrete blanket both within the dam structure and in the tailwater area over a length of 70 m from the buttress toe.

The investigations of water discharges during construction permit the following conclusions to be drawn:

1. The ratio of the cross sections of the sluiceways in the first tier of the dam to the cross section of the opening in the nonramified gallery is approximately constant, varying from 0.72 to 0.73 (see Figure 103).

2. The streamflow regime in the gaps between the buttresses during the passage of temporary water discharges does not present any particular danger, either when a single outlet is operating, or when the outlets in the adjacent buttresses are under operation; therefore, the amount of outlets, as well as their location, should be chosen so as to ensure the required water-discharge capacity [of the outlets].

VNIIG IMENI B. E. VEDENEEV LABORATORY OF HYDRAULIC ENGINEERING

Head: A. G. Averkiev, Candidate of Technical Sciences, Senior Research Worker

REDESIGN OF EXISTING, AND DESIGN OF NEW TYPES OF WATER-OUTLET STRUCTURES OF HYDRO DEVELOPMENTS

Responsible for Research: M. E. Faktorovich, Candidate of Technical Sciences, Senior Research Worker

The main purpose of this investigation was to apply the results of the VNIIG investigations on efficient methods of energy dissipation to the design of new types of spillway structures.

A critical examination of the engineering requirements for the operation of spillway structures and of the mechanism of energy losses in various types of energy dissipators, led the researchers to the conclusion that spillway design could be improved: 1) by using energy dissipators which divide the nappe into a number of separate streams with the formation of eddy zones, and 2) by using the spillway structure itself to dissipate a part of the energy, without impairing the strength and stability of the structure.

Accordingly, a number of spillway schemes were considered and one of these was designed and checked experimentally. This was the so-called "spillway dam with multiple-jet outlet and energy dissipation on a slotted spillway face."

This type of dam (Figure 104) consists of massive continuous piers (1), water outlet bays (2), and high-pressure water outlet galleries (3) passing through the piers. On the upstream side, the water-outlet gallery can be closed by a gate (4), and on the downstream side, it is close to the water-discharge distributing shaft (5) between the body of the dam and the slotted

* [Obvious misprint: should read 1600, 70, and 30 m^3/sec. The term specific discharge frequently encountered in Soviet literature, and expressed in m^3/sec actually means discharge rate per running meter of total front of the dam].

overflow (6). The latter is either a slotted slab (7), or a set of beams (8), the spaces between them constituting the spillway slots. The frame of the slotted overflow should be firmly fastened to the dam piers. This type of dam may be supplemented by an overflow crest if this is required for the passage of ice or floating debris.

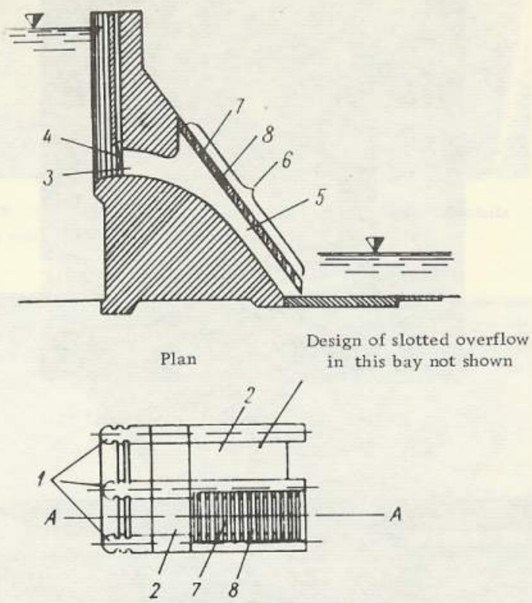


FIGURE 104. Spillway dam with multiple-jet energy dissipator on a slotted overflow

1-piers; 2-spillway bays; 3-high-pressure water outlet gallery; 4-gate; 5-inclined water-discharge distributing shaft; 6-slotted overflow; 7-slab of slotted overflow; 8-beams.

To verify the feasibility of this type of dam, the laboratory prepared a schematic plexiglass model (Figure 105). The model tests confirm the high degree of energy-dissipation in the slotted overflow. Thus, with a kinetic coefficient $F = 50$, the second depth of the hydraulic jump (at the tailwater section) is reduced by at least 35% as compared with the depth of a freely formed hydraulic jump in the absence of energy dissipators. Figure 106 illustrates the general streamflow pattern.

In cases where the junction between the headwater and tailwater occurs under difficult conditions with only partial operation of the spillway, this type of dam may be supplemented by a longitudinal water-outlet gallery. The late I. I. Weits, of the VNIIG, has proposed a forced water discharge through a longitudinal gallery; according to his proposal, only one spillway bay is opened, and the discharged stream, flowing through the longitudinal gallery, is distributed within the structure along its whole front, and emerges at the tailwater with a correspondingly smaller specific discharge. Experimental tests of this improved water-discharge scheme showed promising results.

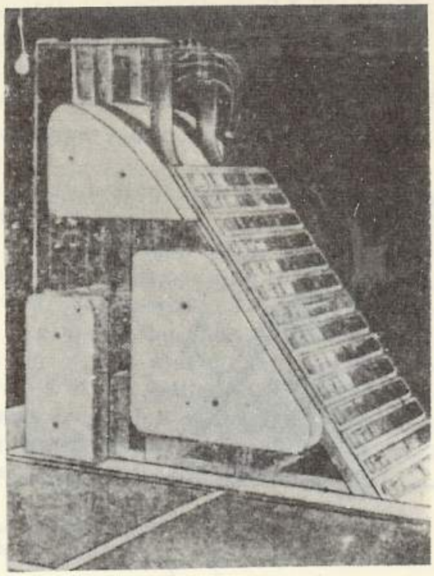


FIGURE 105. Schematic dam model

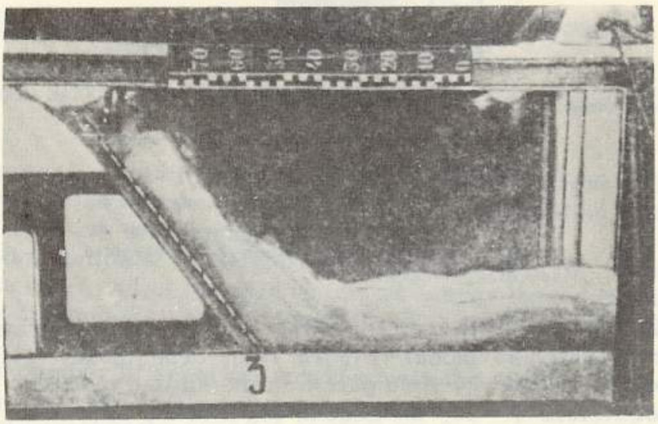


FIGURE 106. Streamflow pattern in the dam model

Among the other schemes considered, the design of a spillway dam provided with energy-dissipating baffles mounted on the spillway face (Figures 107, 108) may be mentioned. Such an arrangement provides for a repeated dispersion of the falling water nappe over the spillway face and collision between the separate streams either in the air or on the spillway face.

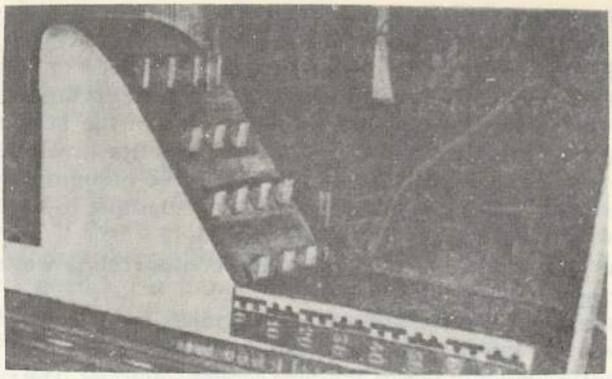


FIGURE 107. Spillway dam model with energy-dissipating chute baffles mounted on the spillway face

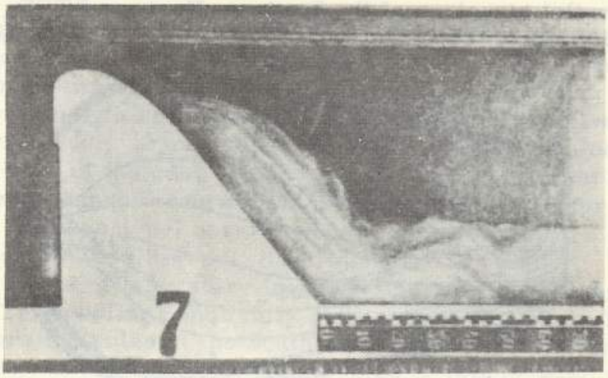


FIGURE 108. Stream separation and collision over the spillway face

FIELD VERIFICATION OF RESULTS OF LABORATORY INVESTIGATIONS ON HYDRAULIC PHENOMENA AT HYDRO DEVELOPMENTS AND STRUCTURES

Section I

Responsible for Research: G. N. Lapshin, Candidate of Technical Sciences, Senior Research Worker

The hydraulic laboratory [of the VNIIG] has been conducting for several years scientific research on the design, construction, and operation of hydro structures in thermal power stations and industrial plants.

This research covered mainly: a) water intakes; b) cooling ponds; c) water-outlet structures; both conventional and new modeling methods were used.

Laboratory investigations were also carried out in recent years on natural and mechanical draft cooling towers.

On the basis of these investigations, the laboratory modified and sometimes completely changed the design of the structures and the general layout. These modifications markedly improved operations and reduced the construction cost.

Most of the structures, investigated at the laboratory, have either been completed or are now being constructed.

In general, such laboratory investigations are based on a series of assumptions schematizing either individual structural elements or certain factors observed in connection with the operation of the structures.

Therefore, field verification of laboratory results involving comparison of model-test results with field data appear to be both useful and necessary. Such a comparison permits the model technique to be refined and the schematic assumptions to be substantiated.

During the years 1957 and 1958, such comparisons were carried out for the following structures:

1) water intake of the Southern-Ural Regional Heat Power Station;
2) cooling ponds for the same station;
3) water-outlet structures of the Cherepet' Regional Thermal Power Plant (RTPP);
4) cooling pond of the same plant;
5) cooling pond of the Barabinsk RTPP;
6) water-outlet structure of the Southern-Kuzbass RTPP;
7) mechanical draft cooling tower of the town of Dneprodzerzhinsk.

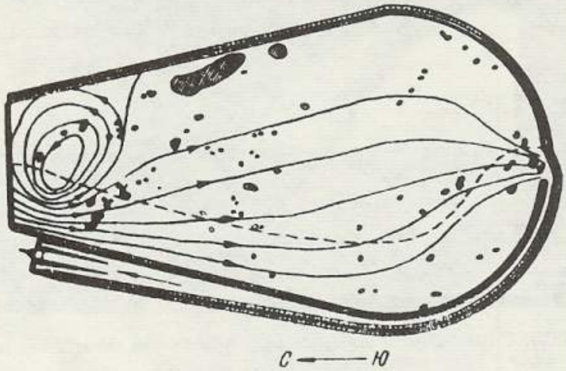


FIGURE 109. Streamflow pattern of the cooling pond

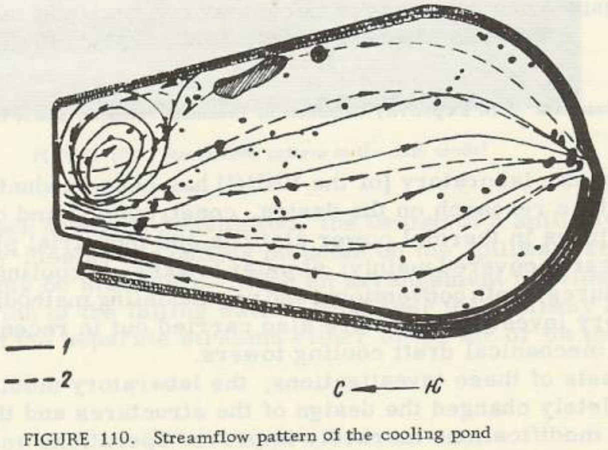


FIGURE 110. Streamflow pattern of the cooling pond

1–according to field-survey data for 1956; 2–according to laboratory data (taking into account the presence of aqueous vegetation).

The laboratory also investigated the cooling tower of the Stalingrad RTPP. A special team of the laboratory investigated on the spot the hydro structures of the Barabinsk, Southern-Ural, Southern-Kuzbass and Cherepet' RTPP, the cooling tower at the Stalingrad RTPP, and the mechanical draft cooling tower of the Dneprodzerzhinsk RTPP. The study was based on the results of field investigations carried out by the LOTEP at the cooling ponds of the Barabinsk, Southern-Ural and Southern-Kuzbass RTPP.

As a result of these studies, the following conclusions were drawn.

1. The streamflow pattern obtained in laboratory tests show in general a sufficiently good agreement with the field data.

A substantial discrepancy was, however, noticed in the flow pattern at the Barabinsk RTPP cooling pond when warm waste water was discharged only through half the discharge front. This discrepancy may be the result of pond bottom overgrowth (see Figure 109) and was eliminated after a detailed study of the silting centers on a schematic aerodynamic model. Figure 110 shows a comparison between field and laboratory streamflow patterns.

2. It is possible to achieve schematic simulation of certain elements of water-discharge structures during model tests, as for instance, replacing the outlets of the water-discharge structures provided with a water-deflecting overflow dam by a simpler design in the form of a spillway (e. g. at the Southern-Ural pond); or replacing the discharge channel located between the original bank of the cooling pond and the island situated within the pond (this discharge channel ensures the overflow of warm water on both sides) by a special (effuser-type) nozzle. The tests were carried out on an aerodynamic model of the Cherepet' RTPP pond.

3. The results of laboratory investigations of the Southern-Ural and Cherepet' RTPP cooling ponds fully agree with field data. Thus, deviation from the operation rules at the Southern-Ural RTPP discharge channel during the passage of flood waters (full opening of two, or even three of the six overflow bays as well as the discharge (0.4% of the probable value) through the completely opened spillway, lead to certain erosion of the banks at the tailwater section (Figure 111). Figure 112 shows graphs of distribution of longitudinal flow velocities (measured under laboratory conditions) at a spillway discharge of 1140 m^3/sec. These graphs agree almost completely with field flow patterns. From the analysis of these graphs it can be seen that the flow velocities in the area of the tailwater adjacent to the spillway dam, were the main cause of erosion of the insufficiently protected river banks.

4. The results of field investigations of the Cherepet' RTPP hydro structures confirmed the conclusions derived from laboratory investigations of the cooling pond and water-outlet structures of this power plant.

5. It is possible to use distortion ratios of 1:4 or 1:6 in aerodynamic models in the case of an even bottom of the cooling pond.

6. For a more exact comparison of the laboratory results on water-outlet structures with field data, it is necessary to carry out field studies on such structures during the discharge of spring floodwaters.

7. Field data on hydraulic-resistance coefficients of cooling towers showed sufficiently good agreement with laboratory data. Thus, the laboratory coefficient ζ for mechanical draft cooling towers with water entrapment arrangement varied between 25 and 42, whereas the actual coefficient varied between 27 and 39.

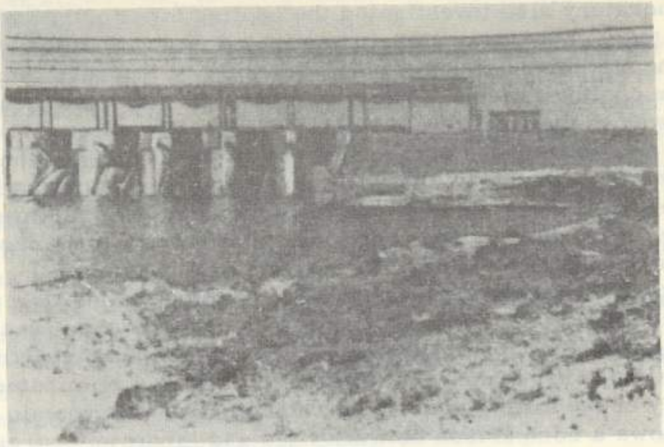


FIGURE 111. Erosion of river-bank stretches along the tailwater section

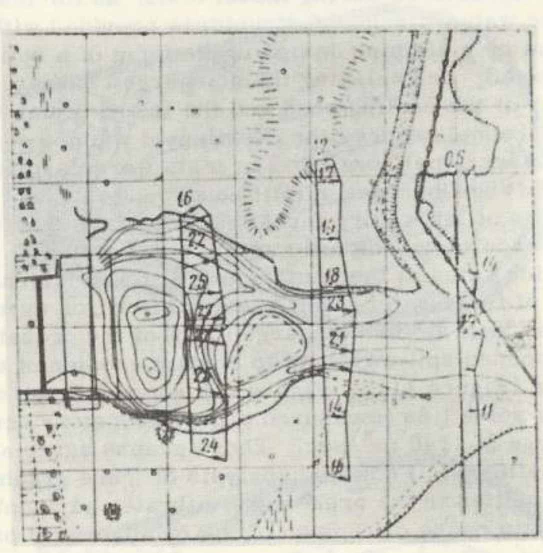


FIGURE 112. Schematic view of river bank-erosion along the tailwater section, as observed in July 1957

8. To ensure more complete correlation between laboratory results and field data, the scope of field investigations should be widened to include additional thermal power plants.

NEW TYPES OF WATER INTAKES (WATER INTAKES OF THE FILTER AND SYPHON TYPES, WATER INTAKES IN MOUNTAIN RIVERS)

Responsible for Research: V. P. Popov, Junior Research Worker

This study, conducted on the basis of scientific results obtained at the VNIIG, imeni B. E. Vedeneev, the GRUZNIIG, ARMNIGGiM, the AZNIIGiM, and the TNISGEI imeni A. V. Vinter, includes construction schemes for water intakes, description of their principle of operation, results of laboratory tests, and of operations of existing structures, hydraulic computation for some structures, as well as testing of recommended designs..

The study includes the description of 13 water intake structures, and consists of two sections.

Section I, description of structures and recommendations regarding

a) the design of water intakes of the filter (combined) and syphon types;

b) the possibility of withdrawal of water from a river without using intake dams;

c) thermal calculations for water-intake protecting structures under conditions of ice passage.

Section II, description of structures and suggestions for the design of water intakes at mountain rivers.

The study provides information for use in the preliminary design of hydro structures, but additional hydraulic investigations are required on certain types of water intakes, and more detailed hydraulic computation methods must be worked out.

HYDRAULIC LABORATORY INVESTIGATIONS OF THE HYDRO STRUCTURES OF THE SANMENHSIA HEP ON THE HWANG HO RIVER DURING CONSTRUCTION AND OPERATION OF THE HEP

Responsible for Research: M. E. Faktorovich, Candidate of Technical Sciences, Senior Research Worker

The construction scheme for the Sanmenhsia HEP provides for the construction, in the first stage, of the base of the spillway dam, with sluiceways for the water discharges during the construction period. The second stage provides for the damming of the river by cofferdams, and the subsequent construction of the power house in the excavation pit, concurrent with the completion of the spillway dam.

During the second stage of construction, water is discharged through the spillway dam crest. During the first construction stage the rated water discharge amounts to 22,700 m^3/sec, dropping to 16,200 m^3/sec during the second stage, as a result of water storage in the headwater. Damming operations are planned to proceed with a discharge of 1000 m^3/sec.

In 1958, the laboratory continued its studies on the hydraulic conditions at the development during construction and operation, and prepared recommendations for ensuring the reliable and economic operation of the hydro development, and also recommended the most suitable designs for the various component elements of the structures. These studies served subsequently as a basis for the preliminary and final designs.

The studies were conducted on two three-dimensional 1:12 scale models: one for studying the conditions of river damming and passage of water discharges during the second construction stage, and the second for studying the operating conditions of the development after its completion.

The problems investigated on the first models dealt mainly with:

1) the design of a scheme for river damming during the construction of the transverse cofferdams of the second stage, including calculation of the volume and the grain size of the materials used for river damming;

2) the study of hydraulic conditions of water passage during the second construction stage, for different designs of the spillway crest;

3) the determination of the water level at the highway bridge piers at the tailwater section of the development;

4) the working out of measures to protect the right bank of the tailwater side, against scouring.

The laboratory, on the basis of investigations, made the following recommendations:

1. The river damming scheme for the second construction stage was approved, with certain modifications suggested by the laboratory. This scheme provides for the construction of the dam rock toe by an original method.

2. The dam rock toe closing the diversion arm of the Shanmenho to be built of rock fill containing at least 15% of stones of 84 to 120 cm and [quarry-cut] tetrahedrons weighing 15 tons each. At the moment of completion of the rock toe, the head drop between the upstream and downstream sections may reach 2.60 m at a rated water discharge of 1000 m^3/sec.

The rock toe of the dam closing the cut through the Shenmentao island was intended to be built with the water outlet of the cut closed. In this case the head drop may amount to 4.0 m.

In view of the destruction of the water-outlet pier in the Shenmentao cut which occurred during construction, closing of the cut is recommended as well as building downstream from the main rock toe an additional rock toe with a length 2/3 of the cut width. The rock toe may be built of rock fill consisting of stones weighing 3 to 5 tons each, and tetrahedrons of up to 15 tons.

In the last stage of river damming, the gates of the spillway are lowered at the diversion arm of the Kweimenho River, thus ensuring dry working conditions for construction of the dam closing this channel. At a rated discharge of 1000 m^3/sec the difference in water level will amount to 7.6 m.

3. If the Hwang Ho River is dammed at a water discharge much in excess of the rated value of 1000 m^3/sec, the head during the construction of the dam will markedly increase, thereby increasing the construction difficulties.

4. In connection with the design of the crest of the second-stage cofferdams, the laboratory calculated the relation of the headwater level and the water level along the cofferdams to the amount of water discharged during the second stage of construction, as well as the distribution of surface streamflow velocities in the constricted area of the river bed.

5. In connection with the planning of alternative ways for speeding up the construction of the hydro project, the laboratory made model tests to work out water level and flow velocity conditions for the passage of water discharges during the construction period, the crest of the spillway dam being raised to a certain elevation.

6. The laboratory recommends the construction of a special water-deflecting dike adjacent to the Changkungshih Island in order to protect the right bank at the tailwater section against erosion.

7. During the maximum water discharges, the hydraulic conditions near the highway bridge are particularly unfavorable (the streamflow reaches a velocity of 14.0 m/sec) a fact which should be taken into account in the design of this bridge. To prevent any possible damage to the bridge roadway, the laboratory suggested construction of a special stream deflector.

The investigations on the normal operating conditions of the hydro development covered the following aspects:

1) the establishment of a general flow regime both for the headwater and the tailwater section for different operation regimes;

2) the study of conditions of stream junction between the headwater and tailwater sections, in order to select optimum junction conditions and protection of the tailwater section;

3) the establishment of the most suitable design for the tailwater channel of the HEP.

The problems in 1) were investigated on a movable bed model. The bed material consisted of glass particles whose sizes corresponded to the prototype, i. e., to individual rock blocks making up the foundation of the tailwater side of the dam. The glass particles were arranged in the model in strict conformity with the natural, columnar structure of the rock foundation. Such a model of the erodible area of the tailwater section permits an approximate evaluation of the extent of erosion in this area.

The results of the experimental investigations may be summed up as follows:

1. The method of energy dissipation by means of an upturned bucket is not the most suitable under conditions of construction and operation of the Sanmenhsia HEP and should be replaced by a smooth junction between the spillway face of the dam and the river bottom at the tailwater section.

2. In order to remove the scouring zone as far as possible from the protective structures of the tailrace, it is advisable to construct, in the area of the bottom outlets, a plain apron provided with a water-deflecting sill 3.0 m high.

3. The separating wall [between the power house and the spillway section] should be adequately protected against scour.

4. The pier separating the sections of the dam containing the overflow and bottom outlets should be continued as far as the end of the protective structure of the apron area.

5. To prevent unnecessary backwater pressure at the tailwater section of the HEP, the downstream transverse cofferdam should be dismantled down to the elevation of 270 m.

The results of these investigations, in the form of a report, were submitted to the design institute for use in the design of the Sanmenhsia HEP. The recommendations ensure the proper operation of the structures and also a considerable saving in construction costs.

Due to the modification of the bottom-outlet sill elevation by 10 m (as required by the customer) a great part of the investigation program had to be repeated to suit the new elevation.

VNIIG IMENI B. E. VEDENEEV LABORATORY OF RIVER HYDRAULICS

Head: Professor A. N. Rakhmanov

DETERMINATION OF THE OPTIMUM TYPES OF WATER-ENERGY DISSIPATORS BY THE EROSIVE CAPACITY OF THE STREAM, UNDER TWO-DIMENSIONAL AND THREE-DIMENSIONAL CONDITIONS

Responsible for Research: Professor A. N. Rakhmanov

The evaluation of water-energy dissipating structures by the erosive capacity of the river downstream from these structures is of particular importance for the proper selection of the most suitable type of energy dissipators.

For this purpose the above laboratory carried out in 1958 a series of studies on the erosive capacity of a water stream (considered here as a two dimensional body) downstream from the apron. These studies covered two aspects:

1) the hydraulic jump for the case of a horizontal (smooth) apron with a water-deflecting sill;

2) the hydraulic jump for the case of an apron of pronounced roughness.

The problem under § 2 involved a series of experiments for the determination of operating conditions of the elements of pronounced roughness at high streamflow velocities, which are liable to cause cavitation phenomena.

In addition to the main research program for 1958, the laboratory studied (on the basis of available experimental data) the problem of the total horizontal pressure of the water stream on the water-deflecting sill.

The results of these investigations may be divided into three groups:

1. For horizontal aprons with a water-deflecting sill, the laboratory established in 1958 the basic types of a plane stream (under conditions of hydraulic jump), the general hydraulic regime for such a stream (for a vertical upstream face of the deflecting wall), the formulas for calculating the horizontal and the vertical dimensions of the hydraulic jump, the most suitable location of the water-deflecting sill, and finally, the limits of applicability and most suitable flow conditions for such energy dissipators.

For the particular case of the eddy zones of the hydraulic jump under optimum conditions of its formation, the laboratory found, for a wide range of kinetic-parameter variations, the following relationship:

$$\frac{l_k}{h_{20}} = 4.2 + 0.14\left(\frac{S_c'}{z_{po}} - 1\right), \quad (1)$$

where S_c' = distance from the beginning of the jump to the water-deflecting sill;

h_{20} and z_{po} = depth of water at the end of the hydraulic jump, and its height (with no water-deflecting sill).

For the minimum attainable water depth h_2 at the end of the hydraulic jump (for the case of its critical position and assuming the existence of a water-deflecting sill) the following relationship was found:

$$\frac{h_2}{h_{20}} = 0.35 + \frac{1{,}02}{\sqrt[3]{F_1}}, \quad (2)$$

where F_1 = the kinetic parameter of the streamflow related to the water depth h_1 at the beginning of the jump.

The height c and location of the water-deflecting sill, and, in some cases, its elevation (for a critical shape of the jump) may be found by plotting graphically the relationships $\frac{h_2}{h_{20}}$ and $\frac{c}{h_{20}}$.

Furthermore, the laboratory derived formulas for the following parameters: the length of the first and second surface eddy zones, the length of the bottom eddy zone downstream from the water-deflecting sill, the length of the hydraulic-jump zone downstream from the eddy zone, the maximum water depth h_c in the immediate proximity of the deflecting sill (at its downstream side), the maximum elevation of the free water surface above the downstream apron, the thickness of the intermediate (transition) nappe downstream from the sill, and certain other dimensions of the hydraulic jump, in the presence of a water-deflecting sill.

The study also includes a comparison of experimental data with theoretical hydraulic computations of the water-deflecting sill, as well as velocity and discharge coefficients m and φ for a water-deflecting sill with a vertical upstream face.

For the case of hydraulic jumps, where there is a deflecting sill, the study gives graphs of the average (over a certain time interval) total horizontal pressure P per unit width of the sill, for the critical shape of the hydraulic jump and for some cases of submerged jumps. These graphs show the relationship between $\frac{c}{h_{20}}$ and $\frac{2P}{\gamma h_{20}^2}$, or $\frac{2P}{\gamma h_{\kappa}^2}$ or $\frac{2P}{\gamma h_{20}^2}$ for different values of $\frac{S_c'}{z_{po}}$ and F_1.

For maximum possible average values of horizontal pressure P on the deflecting sill (for a critical shape of the jump) the following relationship is proposed:

$$\frac{2P}{\gamma h_{\kappa}^2} = 0.7\,(\sqrt{F_1} - 1.9)\,. \qquad (3)$$

2. For the hydraulic jump in the presence of an apron of pronounced roughness, the laboratory refined the empirical relationships for the erosive capacity of the water stream downstream from the apron, and derived new graphs for the variation of the erosive capacity of the stream along such types of aprons.

For the same type of apron the study investigated the dependence of the erosive capacity of the stream along the apron on a series of factors such as length of apron, length of its rough portion, type of roughness, height of and distances between peaks,the roughness elements, kinetic parameters degree of submersion of the jump, etc. The problem of the variation of erosive capacity along the rough apron is considered and some quantitative characteristics of this variation are proposed.

All these studies are illustrated by graphs, figures and general explanations.

3. In order to clarify the conditions at the roughness element at increased flow velocity when cavitation phenomena are likely to appear, a series of experiments was conducted which established the pattern of distribution of pronounced roughness in the initial and most dangerous zone of

the apron, for the case both of a distant (undulated) and close (direct) jump at the downstream face of the dam.

Since the investigation program provided only for a limited processing of the experiment results, the final results will be presented together with the results of experiments scheduled for 1959. It is therefore necessary, apart from processing data already obtained, to systematize and prepare for publication data on possible reduction of water depth at the end of the hydraulic jump and on the length of the eddy zones at the jump for different types and sizes of rough aprons. Similar studies should also be initiated for aprons with water-deflecting cutoffs and stilling pools.

The results of the studies for § 1 and § 2 of the above schedule are due to appear in the near future.

HYDRAULIC LABORATORY INVESTIGATIONS OF THE HYDRO STRUCTURES OF THE SANMENHSIA HEP ON THE HWANG HO RIVER, DURING CONSTRUCTION AND OPERATION OF THE HEP

(Investigations of the spillway dam on a plane and semispatial model)

Responsible for Research: M. F. Skladnev, Candidate of Technical Sciences, Senior Research Worker

In 1957 the laboratory started the investigations of a spillway dam on a 1:75 scale model; these investigations were continued in 1958.

In this year the laboratory investigated a spillway dam provided with bottom outlets and water-conveying galleries with gravity flow, at two elevations of the inlet sill of the water outlets.

Since these investigations were carried out during the preparation of the preliminary and the working designs for the HEP, the laboratory paid particular attention to the following problems:

1) development of a suitable design for the water passageways of the bottom and temporary (overflow) outlets, so as to ensure elimination of undesirable vacuum phenomena;

2) detailed study of the discharge capacity of the bottom and overflow outlets;

3) selection of the optimum flow junction between the water nappe and the tailwater, under conditions prevailing at the Sanmenhsia hydro development, design of water-deflecting structures, and exact determination of loads acting on the apron, separation wall, and deflecting sill.

In view of the particular geological structure of the HEP foundation, the necessity of carrying out construction work and filling the storage reservoir in several stages, and the difficult hydraulic regime of water discharges during construction, the laboratory suggested erecting a spillway dam with smooth flow junction between the bottom and overflow outlets and the tailwater apron. In the area of the bottom outlets the design provides for the erection of a deflecting sill 1.5 to 3 m high with an upstream-face slope angle of 35°. By means of such a sill the stream is thrown off the dam to a distance of 80 to 100 m, thus ensuring a stable hydraulic regime in the tailwater section, and preventing the rock foundation of the apron from being eroded. The absolute amount of erosion was determined on the model by means of gravel soil, simulating the actual rock foundation. The grain size of gravel corresponded to the size of the actual individual rocks which make up the river bottom at the tailwater section.

The investigation results in the form of a report were submitted to the institute designing the Sanmenhsia HEP.

HYDRAULIC LABORATORY INVESTIGATIONS OF A SYPHON SPILLWAY

Responsible for Research: M. F. Skladnev, Candidate of Technical Sciences, Senior Research Worker

The investigations were carried out on a water-storage concrete dam, an earth fill dam, and a syphon spillway for the passage of normal (operational) water discharges up to 200 m^3/sec.

The crest of the syphon spillway is located at the normal pondage level (161.5 m). The spillway consists of eight conduits, 2×1.8 m in cross section, separated by piers.

The particular conditions under which a syphon spillway operate, required special laboratory research. The main purpose of these studies was the development of special measures ensuring proper priming and discharging of the syphon with particular attention to wintertime operation; determination of pressure distribution over the spillway surface and the syphon contours, as well as of the discharge capacity of the syphon spillway; investigations on the possibility of passage of floating debris through the syphon and on the proper operation of the previously designed energy-dissipating structures in the tailwater section of the spillway.

These problems were studied on a 1:15 scale model which simulated a section of the syphon together with its adjoining upstream and downstream structures.

The experimental technique applied in these studies was that used at present in the study of hydraulic processes developing under the action of gravity; the basic dimensions established on the model being converted to prototype conditions according to the gravity modeling law (excluding the time factor).

The variant of syphon spillway developed by the design institute was found to be unsuitable to the priming conditions of the syphon; the vacuum built up at the spillway face of the water-outlet leg of the syphon (at a discharge of 28.4 m^3/sec) amounts to 7.3 of water gage; the construction of an open discharge regulator and location of the outlet edge of the syphon funnel in the tailwater sections ensures troublefree operation of the syphon during the winter season.

To improve the operating conditions of the syphon spillway, the laboratory suggested a series of structural modifications in the design of the syphon spillway (see Figure 113). In particular, in order to ensure quick priming at headwater levels close to the NPL, the laboratory recommended the provision, at the spillway face of the system, of a deflecting sill at an elevation of ~158.0 to 158.8; the location of the outlet edge of the syphon funnel at an elevation ~156.3, with a submersion of 0.4 m below the tailwater level; the provision of an air inlet conduit for 0.20 m in diameter for the discharge regulator, fitted at its end with a regulating coupling intended for varying (if necessary) the height of the air conduit inlet edge. Furthermore, in order to ensure normal syphon operation during winter, the laboratory recommended providing the discharge regulator with a closed chamber and

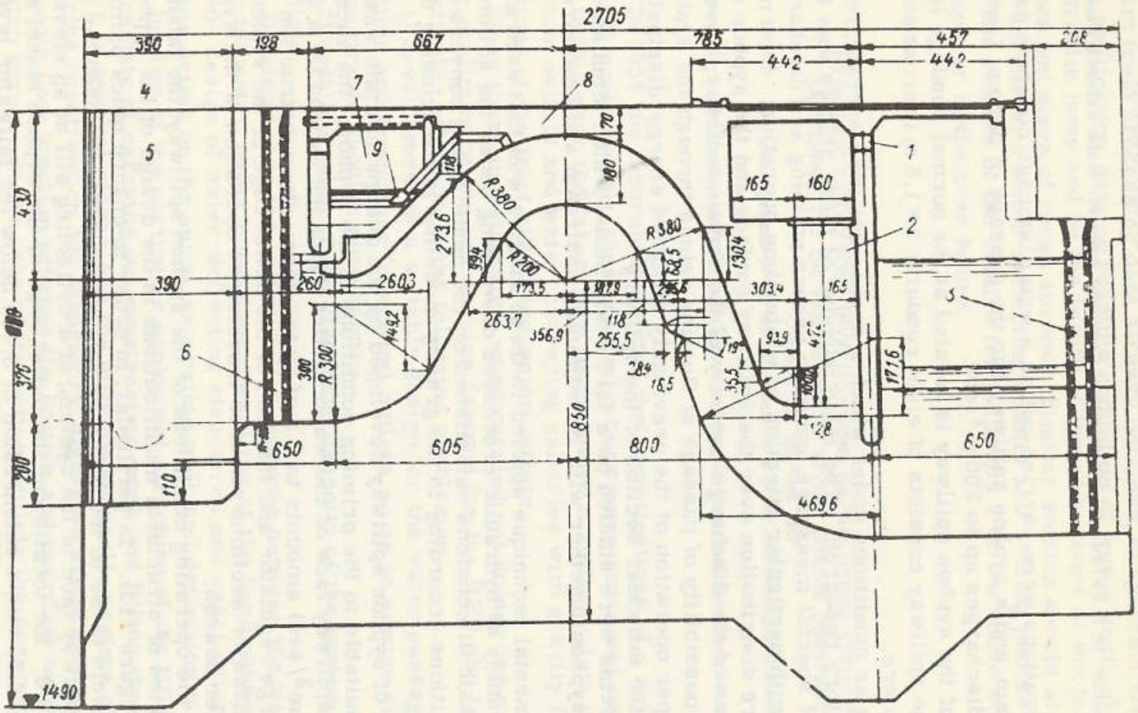


FIGURE 113. Syphon-spillway design variant as proposed by the laboratory

1–inlets for air access to the frost pit; 2–frost pit; 3–guide slots for repair gates; 4–air conduit for elimination of vacuum; 5–air inlet into the syphon chamber; 6–guide slots for repair gates; 7–chamber of discharge regulator; 8–air conduit; 9–regulating coupling.

equipping it with heating facilities. Water inlet into the regulator chamber from the headwater section occurs through openings located 1.5 m below the NPL. A special frost pit (shaft) of the closed type is provided to prevent the downstream face of the syphon funnel from freezing.

With the above modifications, which are intended to improve syphon operation during the winter, the revised design of the syphon spillway has the following characteristics: priming and operation of the syphon, with automatic regulation of the water level according to the storage-lake level, occurs at headwater levels very close to the NPL; maximum value of vacuum within the syphon does not exceed 4.9 W. G.

These recommendations were taken into account in the preparation of the working design for the syphon spillway.

VNIIG IMENI B.E. VEDENEEV SEEPAGE LABORATORY IMENI N.N. PAVLOVSKII

Head: Prof. V.I. Aravin, Doctor of Technical Sciences

A TECHNIQUE FOR USING RADIOACTIVE ISOTOPES IN LABORATORY AND FIELD STUDIES OF SEEPAGE, PIPING, AND SILT-DEPOSITION PHENOMENA

Responsible for Research: V.I. Aravin, Doctor of Technical Sciences

Research by: O.N. Nosova, Junior Research Worker

The laboratory carried out, from January 1 to December 15 1958, the following research work:

1. On the basis of Soviet and foreign literature, the laboratory prepared a survey of the use of radiotracers in the study of seepage processes. Much attention was paid to the development of a suitable technique for measuring radioactive preparations of different degrees of radioactivity. The laboratory also carried out a review of theoretical and experimental studies on the flow of solutions in porous media.

2. The laboratory designed a special unit (Figure 114) for radiotracer studies on seepage processes. The component elements of this unit (water-overflow container, automatic recorder of pressure variations in the seepage water, etc.) were subjected to a careful check over a period of several months.

3. During construction of the radioactive-research laboratory, the quality of construction work was continuously checked and expert advice rendered to the designer where necessary.

4. The working staff was trained in the use of radiometric instruments.

5. The installation of the laboratory equipment was carried out according to schedule.

Representatives of various organizations (VNIITO, SANIIRI, etc.) were consulted on problems of planning and designing radioactive-isotope laboratories.

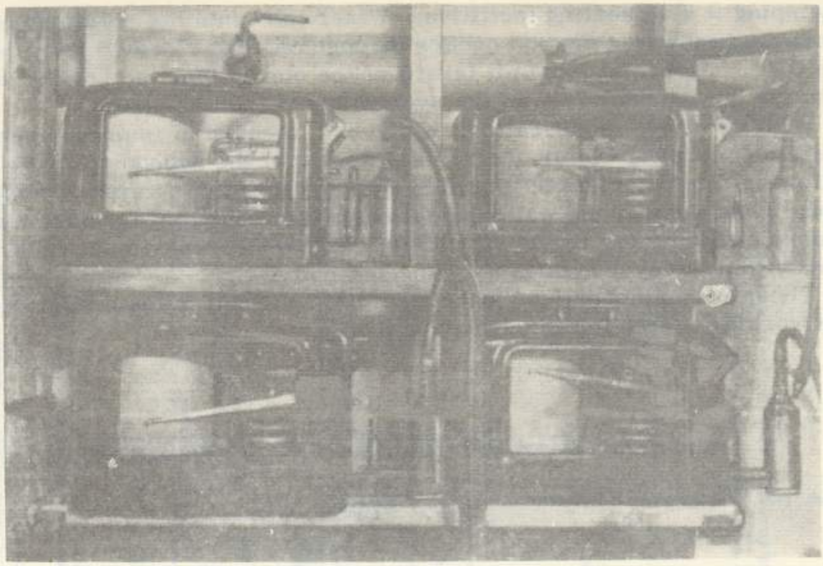


FIGURE 114. Experimental unit for testing seepage phenomena by means of radioactive isotopes

INVESTIGATION OF MECHANICAL AND CHEMICAL PIPING IN ROCK AND SAND-GRAVEL SOILS

Responsible for Research: Prof. A. N. Patrashev, Doctor of Technical Sciences, Honored Scientist of the RSFSR

Responsible for the Research on "Mechanical Piping": M. P. Pavchich, Senior Engineer

Responsible for the Research on "Chemical Piping": G. Kh. Pravednyi, Candidate of Technical Sciences, Senior Research Worker

The study consists of two sections.

Section I. Mechanical piping

In order to establish the piping characteristics of soils and criteria for the design and operation of hydro structures, the research team carried out a series of laboratory and field tests as well as theoretical computations, the results of which were reported to the All-Union Conference on Applied Mechanics and Mathematics.

These investigations covered the following problems:

1) development of methods for determining the piping stability of sand-gravel soils for a wide range of grain-size variations;

2) determination of conditions under which piping phenomena would not affect the stability of the soil and of the structures erected on it;

3) development of methods and test equipment for field determination of the piping resistance of soils.

In determining the seepage and piping characteristics of soil having a nonhomogeneous structure, the seepage laboratory applied the method of comparing the grain-size curve for a natural soil mixture with the curve for a soil mixture of the same composition not subjected to piping, but touching the first curve at two points.

For determining the piping characteristics of a homogeneous soil, the laboratory is working on a test unit and a method based on the measurement of electric resistance of the soil.

In 1958 the laboratory obtained the following results:

1. From tests on soils not subjected to piping phenomena, a new relationship was obtained for the determination of the diameter of soil pores and of the seepage coefficient, taking into account the coefficients of nonhomogeneity, and of nonuniformity of grain-particle distribution in the soil.

2. A relationship was obtained for determining the interlayer coefficient, taking into account the nonhomogeneity coefficient of the filter.

3. The following computation graph was plotted

$$\frac{d_0}{d_{17}} = f(n, \eta),$$

where d_0 = average diameter of soil pores;

n = soil porosity in fractions of unity;

η = coefficient allowing for the difference in grain sizes of the soil.

4. It was proved that it is necessary to take into account the coefficient of nonuniformity of grain distribution within the soil layer; the soil investigations were carried out for a wide range of grain variation.

5. The laboratory designed and constructed a test unit, which was checked experimentally to determine the initial entrainment of minute soil particles by the seepage flow.

6. The laboratory devised a new type of sand trap for taking samples from soil particles carried by the seepage flow.

Section II. Chemical piping

The problem of determining the rate of dissolving and leaching out, by the seepage flow, of soil salts from the rock foundation, the dam body and the structures adjoining the river banks, is of particular significance for the erection of hydro structures on soils containing water-soluble salts.

The investigations had as their aim:

1) to develop methods for calculating chemical piping in sand-gravel soil and rock cracks;

2) to determine the permissible limits for chemical piping in the above types of soils.

The following work was carried out during 1958:

1. A special test unit was designed and constructed for the study of chemical-piping processes (Figures 115 and 116).

2. A technique was developed for the study of chemical piping by the electric-resistance method.

3. A series of rating tests was carried out for determining the salt concentration of soil samples in cases where seepage was present, and also where it was absent.

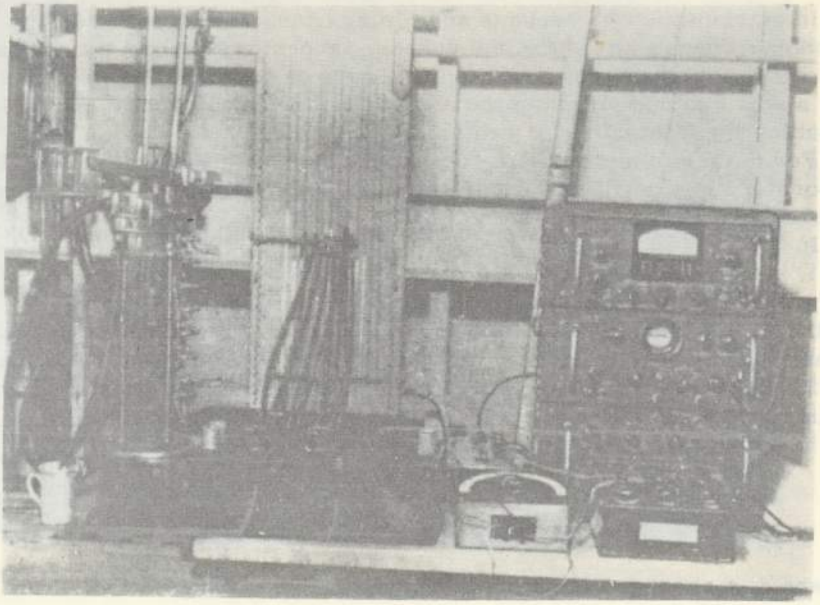


FIGURE 115. Experimental unit for studying the processes of chemical piping

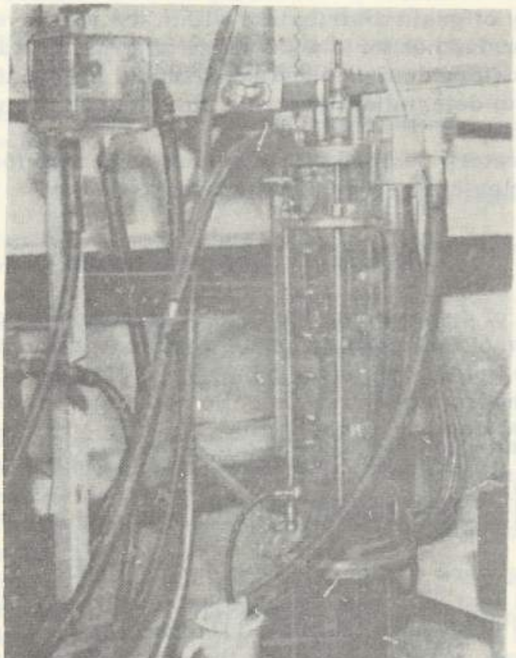


FIGURE 116. Test unit for the study of chemical-piping processes

4. A theoretical solution was obtained for the problem of determining chemical piping in salty sand-gravel soils with uniform seepage.

For such cases the basic formula for leaching processes has the following form:

$$\frac{\partial c}{\partial t}+\frac{(c_0-c)\,D_\Phi}{\gamma}\left(\frac{\partial^2 c}{\partial x^2}+\frac{c_0-c}{a d_0^{\,2}}\right)=0,$$

where c = saturation of the seepage stream with leached-out salt;
c_0 = maximum salt saturation of the seepage stream;
D_Φ = diffusion coefficient;
d_0 = diameter of soil pores;
γ = density of solution;
a = a distribution coefficient, depending almost entirely on the seepage coefficient.

As has already been mentioned, the results of these investigations were reported to the All-Union Conference on Applied Mechanics and Mathematics.

DIRECTIVES FOR APPLICATION OF METHODS IN SEEPAGE CALCULATIONS FOR THE LOWERING OF GROUND-WATER TABLE

Responsible for Research: L. N. Pavlovskaya, Senior Engineer (VNIIG)
V. M. Shestakov, Candidate of Technical Sciences, Senior Research Worker (VODGEO)

The above directives were worked out by the VNIIG imeni Vedeneev jointly with the VODGEO, on the basis of seepage calculations for the lowering of the ground-water table; these directives were carried out at both institutes and were supplemented by other studies in this field.

The directives deal with the methods of calculating ground-water table lowering in horizontally stratified soil where the water table can be assumed to be horizontal and the soil is homogeneous or nearly homogeneous.

The directives were worked out for practical application in ground-water drainage work carried out in the excavation pits for hydro structures. The same methods have been used in seepage calculations in connection with the lowering of the ground-water level in excavation pits for civil engineering and industrial structures.

The directives contain 4 chapters:

Chapter I. "General problems of seepage calculations for ground-water table lowering" deals with the following:

a) a dry season streamflow is taken as a typical example in the determination of the characteristics of equipment for lowering the ground-water table, while, as an example of checking calculations for the determination of the additional reserve equipment, the case of initial dewatering, floodwater, etc. is considered;

b) classification of water-table lowering according to hydrogeological conditions in the excavation area and to feeding conditions of the seepage flow;

c) classification of water-table drainage equipment according to boundary conditions prevailing in the area;

d) basic methods for seepage calculations of water-table lowering: the method of summation of water currents and the method of equivalent seepage resistances.

Chapter II. "Seepage calculations for equipment for ground-water table drainage under steady-seepage conditions." This chapter deals mainly with:

a) calculation methods for the case of dry-season flow; calculation methods of water-table drainage by single wells, based on the principle of summation of water currents (the Forchheimer method); (a calculation example is given);

b) calculation method for lowering the ground-water table by a system of contour wells to a given contour in plan (first type of boundary conditions); calculations are made according to the flow-line limits of the flow net plotted for a system of equivalent trenches replacing actual wells; within each flow-line limit, the flow is reduced to a two-dimensional stream, which makes it possible to use the formulas for infinite rectilinear wells; a calculation example is given;

c) water-table lowering calculations for a system of contour wells with given drainage capacityof wells (second type of boundary conditions). The whole contour of drainage wells is divided into several rectilinear areas, each characterized by a constant specific-discharge capacity; the whole system of wells is considered as a system of interacting lines of equal inflow. The fall of the level at each point is defined as the sum of level falls in each separate line of equal inflow. This section of the chapter is also illustrated by a calculation example.

Chapter III. "Seepage calculation of water-table lowering under conditions of nonsteady seepage for the case of initial dewatering" deals with:

a) calculation methods for initial (first) dewatering of soil under conditions of gravity seepage flow and of lowering the ground-water table under conditions of nonsteady seepage flow. Methods are given for primary drainage by single wells during dry-excavation work. These methods are similar to the methods used for steady seepage flow. The drainage capacity of each well is assumed to be constant or to vary in stages;

b) the calculation of primary drainage of soil by means of a contour system of wells for the case of hydraulic excavation when deep-well drainage is combined with open-trench drainage. The calculation method is based on the assumption of nondeformable flow lines. Calculation is made according to a flow net plotted for a steady seepage flow with a given constant level lowering at the excavation contour in plan. Within each area of the flow net, the stream is reduced to a two-dimensional flow. Such a simplifying assumption permits the use of the formulas for infinite rectilinear wells of constant drainage capacity for nonsteady seepage flow. At the boundary of the area corresponding to the area of the excavation contour, the lowering of the ground-water table is assumed to vary linearily with time. The method is illustrated by a calculation example;

c) description of methods for the calculation of primary drainage by a contour system of wells for dry excavation of the construction pit. For each area of the well contour in the plan are given the specific drainage capacities assumed to be constant or to vary gradually in time. The well system is considered as a system of interacting lines of equal inflow. The

calculation methods for a nonsteady seepage flow are similar to the methods for steady seepage. A calculation example is described;

d) methods for calculating the discharge time of the ground-water table under conditions of nonsteady flow; the method is illustrated by a calculation example.

Chapter IV. "Seepage calculations for ground-water drainage under nonsteady seepage flow for the case of flood and flood damages" deals with problems of drainage under flood-runoff conditions. It is assumed that, during flood runoff, the ground-water table may be lowered to a lesser extent than in dry-season runoff: therefore, the calculation is made at first on the assumption that the drainage unit is operated during flood runoff with the same capacity as in the dry season. The amount of water-table lowering is defined as the algebraic sum of level drop during dry-season flow and the level rise during flood over the dry season level, the excess being determined by the formulas of two-dimensional nonsteady seepage for floodwave propagation. Should the drop in level turn out to be less than admissible, a special flood-runoff margin should be provided for. The capacity of stand-by drainage units during flood runoff is determined, for nonsteady seepage flow, resorting to the solution of a system of lines of equal inflow, or of a flow net. The method is illustrated by a calculation example.

The chapter also describes methods for calculating the time required for flooding the excavation net as a result of damage to the drainage system which, it is assumed, form a system of single wells or a system of lines of equal inflow. The calculations assume the damages to have occurred during dry-season flow, and the drainage units to have been switched off simultaneously. The method is also illustrated by calculation examples.

Chapter I was written by VNIIG jointly with VODGEO, chapter II by VODGEO, III and IV by TsNIIT (several paragraphs jointly with VODGEO).

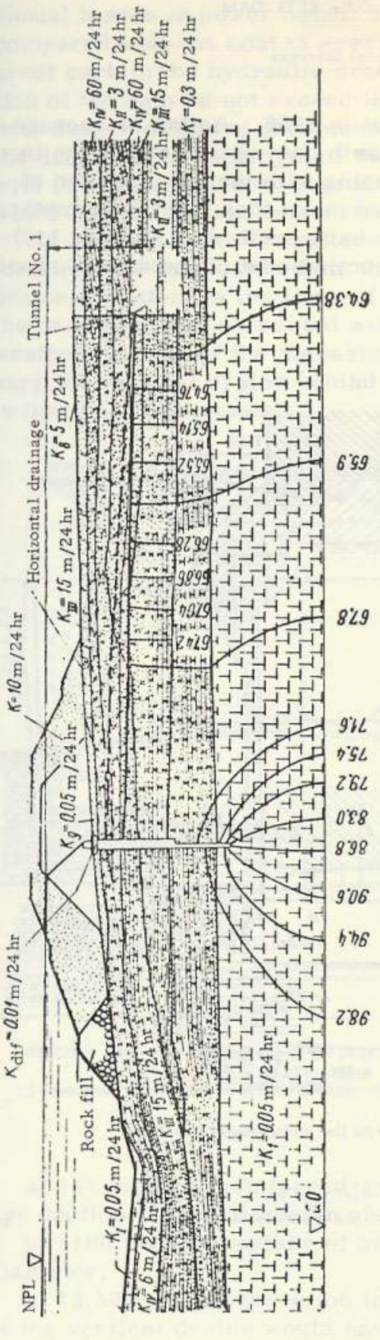


FIGURE 117. Seepage investigation in the body and the foundation of the left bank earth-fill dam of the Bratsk HEP

INVESTIGATION OF SEEPAGE IN THE BODY AND FOUNDATION OF THE LEFT-BANK AND RIGHT-BANK DAMS OF THE BRATSK HEP AND IN THEIR STRUCTURES ADJOINING THE BANKS AND THE CONCRETE DAM

Responsible for Research: N.I. Druzhinin, Doctor of Technical Sciences

The above investigations were carried out in 1958. As far as seepage in the body and foundation of the left-bank earth dam is concerned, the investigations proved the possibility of dispensing with the arrangement of a horizontal conduit drainage over a length of more than 700 linear meters in the downstream toe of the left-hand river bank earth-dam (Figure 117). Such measures permit savings in the construction cost of the dam of about 230,000 rubles.

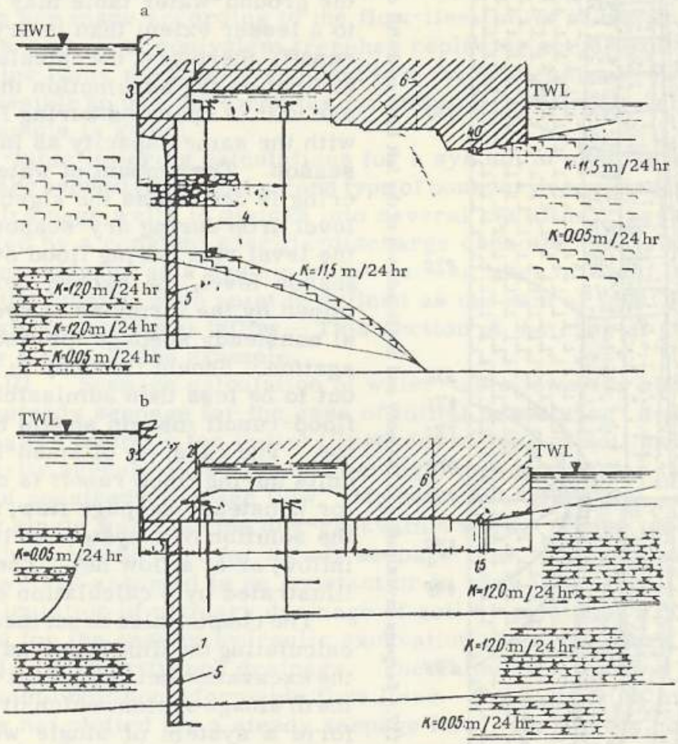


FIGURE 118. Determination of uplift pressure on the underground structures of concrete dams and powerhouses of the HEP

1–contours of expansion joint; 2–dam site; 3–upstream face; 4–drainage wells; 5–grout curtain; 6–line of junction with the excavation pit for the powerhouse.

The study also proved the insignificant effect of a grout curtain (Figure 117) on the variation in seepage discharge and hydraulic gradients. The annual losses in power output are negligible (a fraction of one per cent) compared with the cost of erection of a grout curtain. In the absence of a grout curtain the hydraulic gradients of seepage flow in the body and foundation of the dam do not exceed the allowable limits; hence, there is no imminent danger of piping phenomena either in the body or in the foundation of the dam. On the strength of these facts, the institute suggests to dispensing with the arrangement of a grout curtain on the left-bank earth dam, which would lead to savings in construction cost, of more than 30 million rubles.

As a result of investigations on the seepage flow in four design variants for the right-bank earth dam, the institute recommended a very effective drainage system in the form of a horizontal conduit drainage installed in the dam body, and connected with relief wells driven down to the base of the weathering zone of the diabasic deposits. Such a system renders unnecessary the vertical drains behind the dam and ensures the following savings in the construction cost:

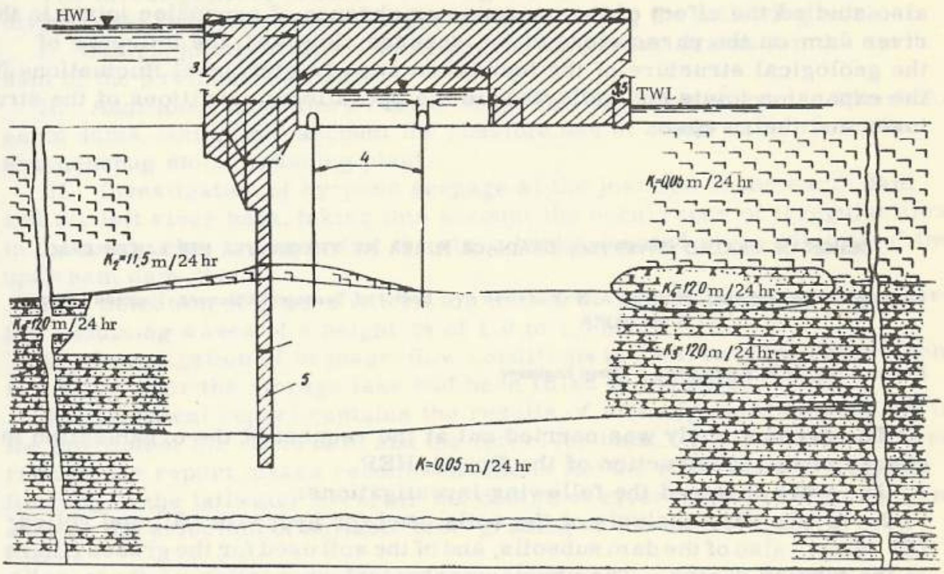


FIGURE 119. Determination of seepage flow in the tailwater section [of the dam]:

1-contours of expansion joint; 2-dam site; 3-upstream face; 4-drainage wells; 5-grout curtain.

a) 147 vertical reinforced-concrete drainage wells having each an average depth of 5 m, a diameter of 1 m and a wall thickness of 0.15 m;

b) 2100 running meters of asbestos-cement drainage pipes, 0.2 m in diameter;

c) 18,500 m^3 of excavation in sandy clay for the trench in which the header of the vertical drains would have been laid ;

d) 560 running meters of drain walls bored in diabasic layers, and arrangement of a drainage backfill, 0.3 m in diameter.

Apart from these works, the laboratory completed in 1958 the study of seepage flow in the foundation of the river bed and river bank concrete dams.

The aim of these studies was to determine the uplift pressure on the underground structures of the concrete dams and the powerhouse, the seepage discharge in the tailwater section, in the expansion joints, in the vertical drainage walls, and in the gallery of the dam and the powerhouse, the hydraulic gradients in the grout curtain, and the velocity parameters of the seepage flow.

The latter were determined for different depths and densities of the anti-seepage curtain (Figures 118, 119) and for different sizes and designs of surface and deep drains.

Several alternatives of operation of the river bed and river bank concrete dams were investigated, the critical interplay of seepage forces in the foundation of concrete structures was established, the optimum sizes for grout curtains determined, and the effect and significance of each row of deep drainage wells of different depth investigated. The research team also studied the effect of the presence or absence of expansion joints in the river dam on the parameters of the seepage flow and the influence of the geological structure of the foundation and of water-level fluctuations in the expansion joints and galleries on the operational conditions of the structures and their repair.

DESIGN OF GRADED (INVERTED) DRAINAGE FILTER AT THE BRATSK HEP EARTH DAM

Responsible for Research: Professor A. N. Patrashev, Doctor of Technical Sciences, Honored Scientist of the RSFSR

Research by: M. P. Pavchich, Group Engineer

The present study was carried out at the request of the organization in charge of the construction of the Bratsk HEP.

The work involved the following investigations:

1. Grain-size analysis of the soils used for hydraulic-fill and rolled-fill dams; also of the dam subsoils, and of the soil used for the graded filters of the tubular drains. The physicomechanical properties of these soils were also studied.

2. Experimental investigation of seepage and of the possibility of mechanical piping both in the body of the rolled-fill and hydraulic-fill dams, and in the quarry materials intended for use in the graded drainage filters located in rocky and earth foundations.

Soil samples, taken in the field, were investigated by means of vertical piping apparatus in which the samples were subjected to loads, and seepage flowed from top to bottom. Measurements were made of seepage discharge, pressure gradients (both local and average), solid load carried by the seepage flow, and settlement, and each soil sample was tested at least three times.

3. As a result of these studies the most suitable size-grading and the number and thickness of the layers of the graded filters could be related.

The operation of the inverted filter was checked in a series of tests by means of a special laboratory-designed unit in which the thickness of the filter layer was almost equal to that of the filters in the prototype.

As a result of these studies, the laboratory recommends the use of a three-layer graded filter.

LABORATORY STUDY OF GRADED FILTERS AND OF SEEPAGE IN THE EARTH STRUCTURES OF THE KREMENCHUG HYDRO DEVELOPMENT

Responsible for Research: G. Kh. Pravednyi, Candidate of Technical Sciences, Senior Research Worker

The study, carried out for the management of the Kremenchug HEP, involved laboratory work on the selection of graded filters and investigation of seepage in the earth structures of the development.

The investigations, conducted during the years 1957 and 1958, were divided into five independent stages. each involving the following work:

I. Design, jointly with the Ukrgidep, of temporary drains for the earth dam of the Kremenchug HEP.

II. Additional laboratory research for the selection of graded filters for earth dams, taking into account the possible use of stone waste from the Kremenchug stone-crushing plant.

III. Investigation of by-pass seepage at the junction of the earth dam and the left river bank, taking into account the occurrence of irregularities in the sandy clay top layers at the river banks, located in the vicinity of the upstream dam shells.

IV. Selection of graded filters for the earth dam drainage system operating, assuming waves of a height $2h$ of 1.0 to 1.7 m.

V. Investigation of seepage-flow conditions in the area of the Taburishchenskii Cape after the storage lake had been filled to the rated level.

The technical report contains the results of investigations on seepage in the junction of the earth dam and the left river bank. On the basis of these results, the report makes recommendations for the prevention of seepage flooding of the tailwater area, for the design of river-bank drainage systems, and for the selection of suitable size-grading of the material for graded filters.

In addition, the following investigations were carried out:

a) reduction in the size of the drainage and the layer thickness of tubular and graded filters with the aim of reducing the volume of drainage work and its cost;

b) studies on filters made of porous concrete.

On the basis of these results, recommendations were made on the building of two-layer graded filters for tubular drains of dams, the material for the filter being stone wastes of the stone-crushing plant. A solution was also found to the problem of reducing the sizes of tubular drains and the layer thickness of graded filters. These measures ensure savings in filtering material, of about 80,000 m^3 and reduction of excavation work by 40,000 m^3, a total saving in costs of about 3 million rubles.

For the design of graded filters of porous-concrete, methods were developed for determining the grading of the crushed rock (gravel) aggregates and a mix with an optimum water-cement ratio was recommended.

The concluding section of the report deals with investigations of seepage conditions in the area of the Taburishchenskii Cape after the storage lake had been filled to the rated level. On the basis of the research results, recommendations are given for the design of a drainage system that would permit the ground-water table to be lowered to elevations provided by the technical specifications.

SEEPAGE INVESTIGATIONS ON HYDRO STRUCTURES OF THE DNEPRODZERZHINSK HYDRO DEVELOPMENT

Responsible for Research: N. I. Druzhinin, Doctor of Technical Sciences

Research by: A. V. Stul'kevich, Senior Engineer

During the year 1958 seepage-flow investigations were carried out:

1) in the river bank junction of the right-bank earth dam;

b) in the body and the foundation of the left-bank earth dam;

c) at the junction between the lower end (head) of the navigation lock and the right-bank earth dam.

As a result of these investigations the institute determined the pattern of the seepage flow and its parameters (discharge, flow gradient, position of seepage line etc.) and made recommendations for the construction and location of the dam and the river bank drainage systems.

The institute also recommended dispensing with the construction of deep trenches initially provided for the right-bank earth dam and to use instead a graded filter at the downstream toe of the dam, whose thickness should be greater than the depth of frost penetration. The institute also recommended an open drainage trench at the downstream toe of the right-bank earth dam, between the lock and the original river bank, extending over a length of 220 m along the 60 contour line.

The drainage ditches along the dam and on the river bank should be provided with relief wells having a diameter of 0.2 to 0.3 m and spaced 5 to 10 m; these wells should pass through the clay and loam soils of low permeability and reach the highly permeable underlying sand deposits at a depth of 0.5 to 1.0 m.

The institute recommended the construction of a drainage ditch at the junction between the right-bank earth dam and the original river bank, at the downstream shell of the dam along line D — B (see Figure 120). The drainage ditch is to be filled with a coarse-grained material and it discharges into the main drainage trench of the dam.

This proposed drainage system has economic and operational advantages; it will prevent the seepage of water out of the downstream toe and also the swamping of the area below the dam.

In order to study seepage in the body and the foundation of the left bank earth dam, the institute carried out, on the EGDA simulator, a series of 11 tests under conditions of the two-dimensional problem for heterogeneous soils, and for the following two cases:

a) tubular drains (in working order and clogged);

b) graded filters with an inclined drainage trench (for the flood-plain area of the dam) or with a riprap blanket (for the river bed area).

These investigations led to the following conclusions:

1. In the flood-plain zone, between the concrete dam and the river bed (l = 300 m) a single graded filter does not prevent the seepage flow from penetrating into the area between the earth dam and the water line of the tailwater section.

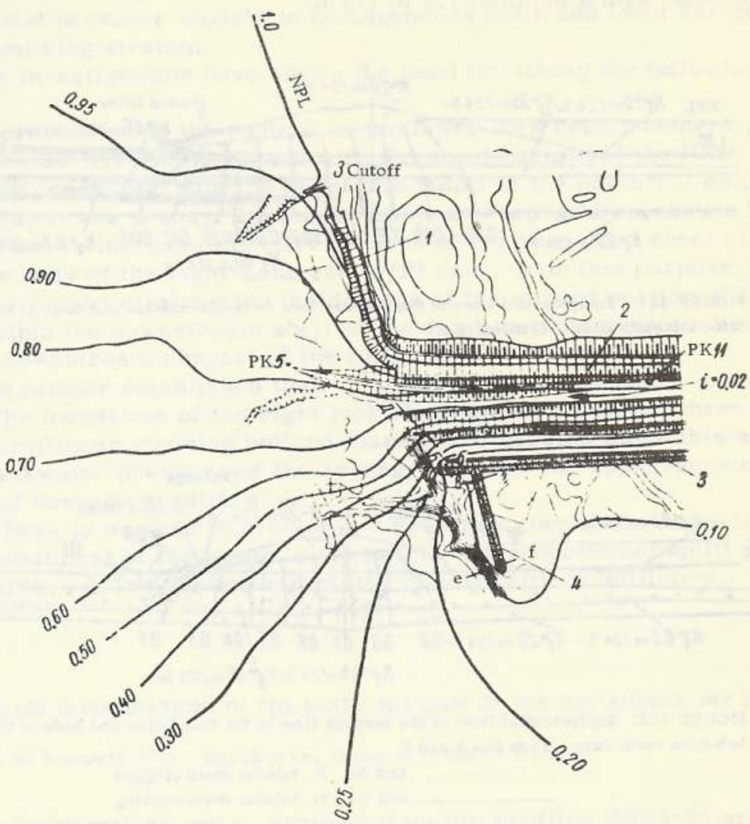


FIGURE 120. Isohypses for ground-water flow in the area of the junction of the earth dam and the right river bank

1 - protected slope; 2 - center line of structure; 3 - dam drainage ditch; 4 - river-bank drainage ditch.

For a complete interception of the seepage flow, the institute recommended the construction, in the immediate vicinity of the downstream left-bank dam toe, of a drainage trench 1.0 m wide and 1.2 to 1.5 m deep, discharging into the river.

2. At the river stretch between points PK 35 and PK 45-60 (over a length l = 1060 m), the investigations were conducted for two alternative elevations of the top crest of the riprap blanket (53.6 and 55.0 m). For both variants, the line of seepage in the downstream toe of the dam turned out to be below the depth of probable freezing (Figure 121); therefore, the variant with

greater elevation of the top of the rock fill is less economical (due to the larger quantity of riprap required), and should be ruled out.

3. In the flood-plain section, between the Dnieper River bed and the Orel River protective dike (at a length l of 3500 m), seepage was investigated for two variants of drainage systems (Figure 122): 1) tubular drains (wells) and drainage trench; 2) graded filter and drainage trench located 2 m from the dam base, with a depth of 1.2 to 1.5 m.

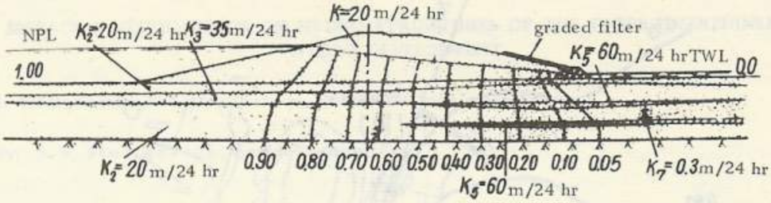


FIGURE 121. Equipotential lines of the seepage flow in the foundation and body of the left-bank earth dam. Test No. 4

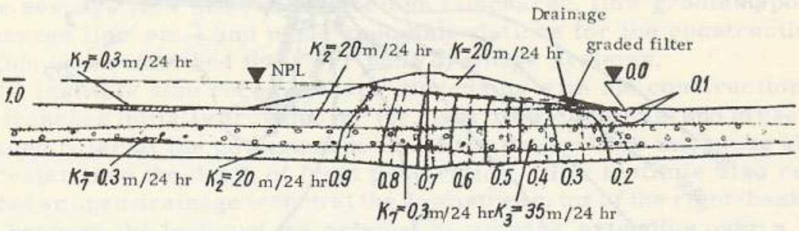


FIGURE 122. Equipotential lines of the seepage flow in the foundation and body of the left-bank earth dam. Tests Nos 8 and 9

.... test No. 8: tubular drain clogged

———————— test No. 9: tubular drain working

The second alternative should be preferred, as it has the following advantages: a) simple and inexpensive construction; b) easy access for inspection; c) reliable operation; d) easy repair or replacement; e) full interception of seepage flow and its diversion into the tailwater section; f) reliable protection of the downstream area against swamping, etc.

The area of the junction between the left-bank earth dam and the concrete abutment of the spillway dam, has also been investigated.

Investigations carried out in the area of the junction between the lower lock head, the right-bank earth dam and the mounting bay of the power house revealed the existence of fissured rocks with weathered pockets filled with kaoline and granite particles. This fact made it necessary to study seepage in this area. The investigations were conducted on three design variants of the junction between the lower lock head and the right-bank earth dam. In all three variants the right abutment of the lower head is embedded in solid rock, and the fissured portions of the weathered pockets under the abutment foundation are covered with a grout curtain.

The first and the third variants do not provide for any tubular drains along the internal face of the curved portion of the abutment, whereas, in the second variant, such a drain is provided. For each of these variants, the laboratory carried out two model tests on the EGDA simulating unit, with and without a 30 m sheet pile cutoff in the soft soil of the body and the foundation of the right-bank earth dam. The tests were conducted on two-dimensional pressure models in homogeneoss soils and for a horizontal water-confining stratum.

These investigations have shown the need for taking the following measures:

1) to provide along the contour of the lower lock head a cement-grout curtain whose left wing connects with the grout curtain of the HEP, the right wing connecting with the sheet pile cutoff of the earth-fill dam;

2) to lower the seepage line of the by-pass seepage flow and its hydraulic gradient and to increase the effectiveness of the 30 m deep sheet pile cutoff in the body of the right-bank earth-fill dam. For this purpose, it is necessary to construct, along the inside edge of the curved portion of the abutment (within the downstream shell of the dam), a tubular drain discharging into the downstream channel of the lock.

It was further established that:

1. The foundation of the right lock abutment and the first three sections of the curvilinear mooring bollard must not be grouted since this would cause backwater pressure of the seepage flow and the appearance of dangerous outlet seepage gradients.

2. There is no need to provide a 6.8 m thick clay apron downstream from the left abutment of the lower lock head for the reduction of uplift pressure in this area. A 3 m thick uncompacted clay backfill is sufficient.

SEEPAGE INVESTIGATIONS IN THE EARTH-FILL DAM OF THE UCh-KURGAN HEP No. 1

Responsible for Research: N. I. Druzhinin, Doctor of Technical Sciences

The investigations were carried out on two profiles (PKO-80 and PK 13-50) of the earth-fill dam.

The PK 0-80 profile was studied for two design alternatives: with and without a sandy clay facing (diaphragm).

For both designs the following parameters were determined, both theoretically and by model tests on the EGDA simulator: location of the drainage system in the downstream slope, the seepage flow-rate, and the position of the seepage lines.

For the second (PK 13-50) dam profile, tests were carried out on the EGDA unit in order to find the optimum sizes of the sandy-clay screen and of the apron, assuming the seepage flow to occur in the pebble layer of the dam body, with a seepage coefficient K_g= 40 m/24 hrs. The other parameters of the seepage flow could then easily be determined.

For the PK 0-80 dam profile, the seepage discharge, hydraulic gradients, and seepage lines were determined by approximation, dividing the seepage area in two sections.

It was proved that the seepage coefficient $K = 10\,m/24$ hrs, adopted by the SAOGIDEP design on the basis of field tests for the gravel-pebble soil of the dam body, was one tenth of the correct value, which led to underestimated theoretical values for the seepage flow in the body and the foundation of the dam, and also to the adoption of an incorrect dam design without any impervious facing.

The water losses Q due to seepage from the storage reservoir amount (according to design data) to $0.205\,m^3/sec$ which, in terms of power output P, amounts to annual losses of 60,000 rubles.

As can be seen from these investigations:

a) The [over-all] seepage discharge Q for a dam without impervious facing and with a seepage coefficient of $K_p = 500\,m/24$ hr) amounts to $20.65\,m^3/sec$ while the annual losses in power output P amount to 6 million rubles.

b) The corresponding figures for a dam with impervious facing or with a facing and an apron are much lower, viz. $Q = 2.83\,m^3/sec$ and $P = 830{,}000$ rubles/year.

The study established the inadequacy of using sandy clay for this facing and the apron — both due to its low cohesiveness and to the existence of considerable seepage gradients ($J = 1$ to 5) along the downstream edge of the impervious facing.

The investigations also revealed the existence of dangerous hydraulic gradients (liable to cause piping) J at the entrance of the seepage flow with the graded filter and the drainage ditch. . Recommendations are made for the reduction of J below the critical value.

The study proved the ineffectiveness of dams built without impervious facings as well as of homogeneous dams provided with a concrete facing.

The study gives optimum elevations for the location of tubular-drainage systems, optimum sizes for the drainage ditch, and data on the composition and design of graded filters.

SEEPAGE INVESTIGATIONS OF EARTH-FILL DAMS

Responsible for Research: N. I. Druzhinin, Doctor of Technical Sciences

The aim of the study was to examine the possibility of using silty sand as a material for impervious facings and cores for composite rock- and-earth-fill dams, and to investigate the feasibility of building homogeneous earth-fill dams of gravel-pebble soils.

The dam site is characterized by: a) high seismic activity (earthquakes up to the 9th degree); b) absence of proper roads; c) absence of a sufficient amount of cohesive (clayey and sandy-clayey) soils and cement which might be used for impervious structures; d) presence of a 15 m thick alluvial deposit in the flood plain and the river bed, having a seepage coefficient of up to 300 m/24 hrs.

The prevailing grain size of the fine-grained silty sands varies from 0.05 to 0.25 mm. Such soils, resembling somewhat lean sandy loam, can only withstand small seepage gradients. The gradients may be reduced by increasing the thickness of the impervious facing, but this would impair the

static stability of the fine-grained silty sand, and this cannot be allowed, in view of the high seismic activity in this region. Therefore, in solving the problem of the suitability of sands for dam facings, it is not only necessary to study the hydrodynamics of the steady seepage flow in the body and the foundation of the dam, but also to examine soil specimens as regards their dynamic stability seepage and piping characteristics. Recommendations should also be made regarding the composition of graded filters which would prevent the clogging of the filter and the occurrence of piping in the fine-grained, silty sand.

In order to study the hydrodynamics of a steady seepage flow under conditions of the two-dimensional problem for structurally, nonhomogeneous, isotropic soils, tests were carried out on the EGDA unit to determine the hydraulic gradients in the anti-seepage elements of the dam, in the pervious foundations and in the contact area between the impervious facing and the ledge rock. Equipotential lines were plotted for the body and the foundation of the dam for the maximum cross section located on the river bed.

The seepage flow was studied for three design variants of the dam:

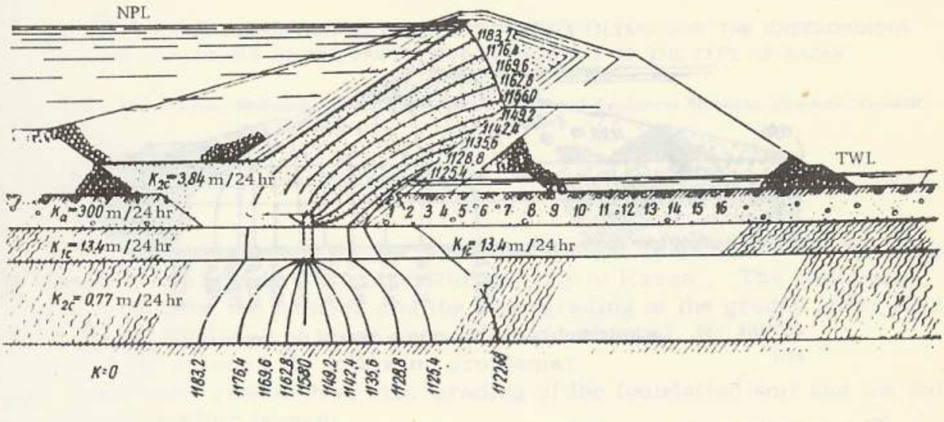


FIGURE 123. Equipotential lines in the cross section of the rock-fill portion of a dam

I — the dam body consists of rock fill and a facing made of sand from quarry No. 2 (Figure 123). This sand has a seepage coefficient K_2 = = 3.84 m/24hrs. Five tests were conducted for the following conditions: absence or presence of a fissured-rock zone and of a concrete cutoff (of varying depth) connecting the facing with the rock foundation; absence or presence of alluvial deposits beneath the sand facing.

II — the dam body is made of soil from quarries No. 6, 2, 7, 8, 9; the dam core which rests on the ledge rock, is made of soils from quarries No. 6 and 2, which have a seepage coefficient K_6 = 0.04 to 0.10 m/hr and K_2 = 0.8 to 3.84 m/24 hr, respectively.

The upstream and downstream shells of the dam are made of gravel soils from quarries No. 7, 8, 9, and rest on the alluvial stratum.

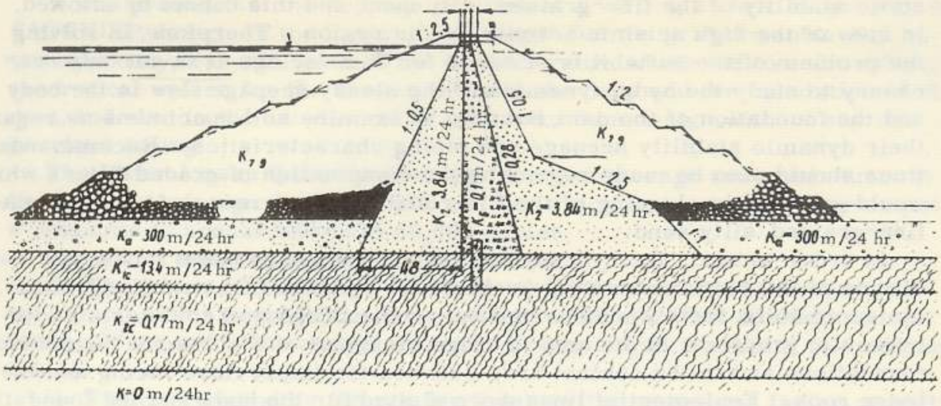


FIGURE 124. Cross section through the dam having a contracted impervious core

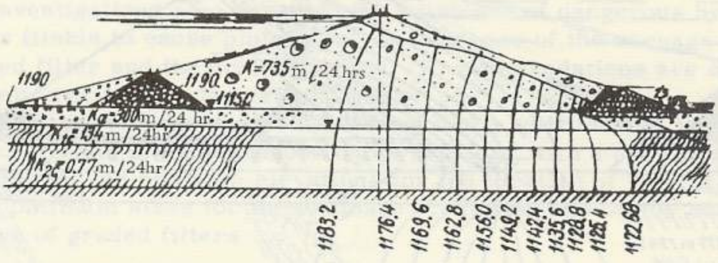


FIGURE 125. Equipotential lines in the cross section of the dam made of gravel soil

Three tests were carried out for the design variant No. II, assuming a fissured rock zone, and for different lengths (l = 0; 12; 18 m) of the concrete cutoff connecting the dam core with the rock foundation.

III — the dam body is homogeneous, being built of gravel soil from quarries, No. 7, 8, or 9. Four tests were carried out for this design variant, three of them assume a fissured-rock zone and differ in the value of the seepage coefficient ($K_{7,\,8,\,9}$ = 735 m/24 hr; $K_{7,\,8,\,9}$ = 3000 m/24 hr; and $K_{7,\,8,\,9}$ = 6000 m/24 hr), while the fourth disregards the perviousness of the dam foundation, while the dam body, with a seepage coefficient $K_{7,\,8,\,9}$ of 6.25 m/24, cuts through the entire depth of the alluvial deposits.

On the basis of these investigations which dealt with the hydrodynamics of the seepage flow, the piping and seepage properties of the soil, its bulk weight, porosity and grain-size distribution, as well as with the analysis of geotechnical conditions at the site and, taking into account foreign practices in the design of high dams made of locally available materials, a series of recommendations were made for the construction of such dams, relating to:

a) the dimensions of the impervious facing and core and the depth of their embedment into the bottom of the ravine and the alluvial deposits of the river bed;

b) the area of junction between the facing and the apron;

c) the values of the maximum hydraulic gradients in the facing and the core and the line of junction between the impervious elements and the ravine bottom;

d) the seepage discharge in the body and the foundation of the dam;

e) the position of the seepage and equipotential lines in the body and the foundation of the dam, in relation to the dimensions, type and location of the impervious elements of the dam;

f) measures for eliminating possible piping phenomena in the soil at the contact zone of the impervious elements, and the washing out of the filler material from the rock fissures;

g) the surface of the seepage area of the ground-water flow into the downstream dam slope, and the determination of optimum sizes of the protective drainage layer required to prevent such a seepage.

LABORATORY INVESTIGATION OF GRADED (INVERTED) FILTERS FOR THE UNDERGROUND TUBULAR DRAINS OF THE PROTECTIVE STRUCTURES OF THE CITY OF KAZAN'

Responsible for Research: Professor A. N. Patrashev, Doctor of Technical Sciences, Honored Scientist of the R. S. F S R.

Research work by: M. P. Pavchich, Group Engineer

The study was carried out for the organization responsible for the Construction of Protective Structures for the city of Kazan'. The basic aim was to determine the number and the size-grading of the graded filter layers required for drainage of these protective structures.

The study covered the following problems:

1) laboratory research of size-grading of the foundation soil and the soil of the graded filter layers;

2) investigation of piping characteristics of foundation soils (in two areas) and of soils for inverted filters;

3) size grading of the soil obtained from local quarries.

The research was carried out by means of special, vertical, and horizontal seepage-testing apparatus under conditions of steady seepage flow.

As a result of these studies, the laboratory recommended replacing the initially-planned, three-layer graded filter by a two-layer filter, and worked out a new design for the drainage filter and a scheme for placing the filter layers, taking into account the conditions prevailing at the building site.

The conclusions and recommendations were approved and forwarded to the organization responsible for the construction work.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR HYDROMECHANIZATION

Head: Professor M. A. Dement'ev, Doctor of Technical Sciences

HYDRAULIC INVESTIGATIONS ON STREAMFLOWS AT THE HEADWATER SECTION OF THE SANMENHSIA HEP CARRYING A LARGE SUSPENDED LOAD OF LOESS *

Scientific Adviser: Professor I. I. Levi, Doctor of Technical Sciences

Responsible for Research: S. I. Gorynov, Candidate of Technical Sciences, Senior Research Worker

The aim of this study was to determine the optimum streamflow conditions required by the design of the Sanmenhsia HEP, which is fed by a river carrying an extremely high amount of suspended load. The problem was investigated on a three-dimensional distorted model (vertical scale 1:100, horizontal scale 1:200). The data obtained were compared with the results of field observations.

The investigation showed that when the bottom flow approaches the storage dam, and is partly reflected from it, the lower, suspension-laden layers rise to the upper, clearer layers of the water. When the storage reservoir was nearly or completely full, the storage coefficient was:

$$K = \frac{Q_1 - Q_2}{Q_1} = 0.43 \div 0.93,$$

where Q_1 = water discharge handled at the model;
Q_2 = actual discharge at the dam.

At a discharge of 6000 to 12,000 m^3/sec, the rise of the silt-laden water layers was such that the thickness of the upper, clear water layers varied between 10 and 20 m.

Investigations of the sediment-carrying capacity of rivers flowing in an easily erodible river bed composed of relatively new loess deposits furnished new data for the problem of increasing the sediment-carrying capacity and for determining it under field conditions.

By comparing the test data with the field results, the following computation formula was obtained:

$$\rho = 430 \left(\frac{v}{v_{cr}}\right)^{4.5} = 14 \frac{v^3}{R^{1.5}},$$

where. ρ = sediment-carrying capacity, kg/m^3;
v = mean flow velocity, m/sec;
v_{cr} = critical velocity at the boundary between normal and rapid flow conditions;
R = hydraulic radius, m.

Figure 126 shows the relationship of the sediment-carrying capacity to the streamflow velocity.

* [On the Hwang Ho River in China]

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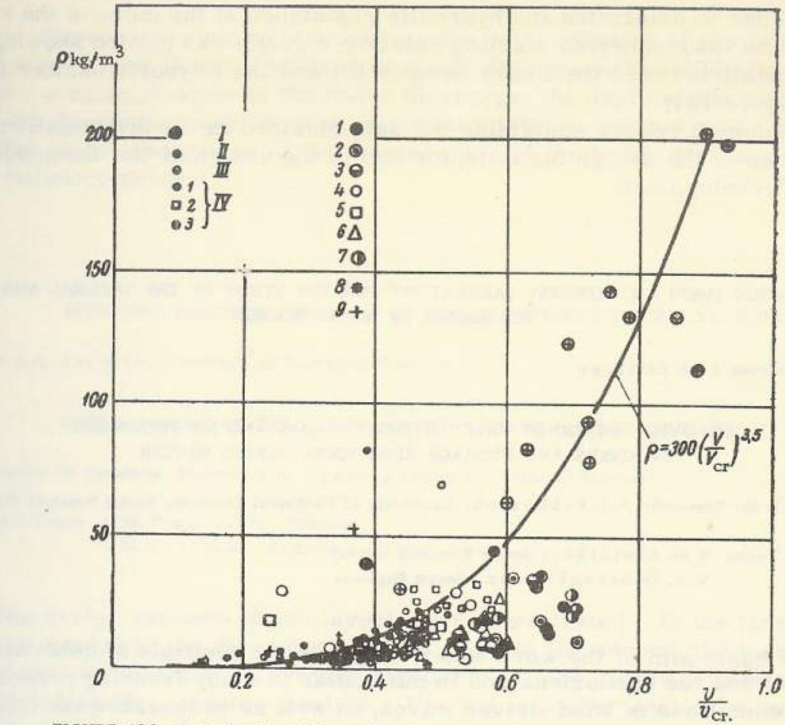


FIGURE 126. Relationship of the sediment-carrying capacity of a river to intensity of the flow

Tests: 1 - VNIIG; II - SANII (laboratory research); III - SANII (field research); IV - channels.

Geographical site: 1 - Tungkwan, Chinchang; 2 - Tsensien; 3 - Kaotsun; 4 - Aishan; 5 - Lokow; 6 - Likin; 7 - Chiangtso; 8 - Sienyang; 9 - Hwansien.

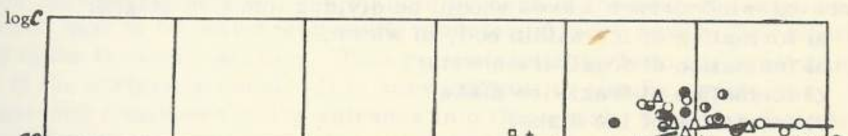


FIGURE 127. Relationship between the Chézy number (C) and the Reynolds number (Re)

In order to determine the hydraulic resistance at the moment the river attains its load-carrying starting velocity, a graph was plotted showing the relationship between the Chézy number (C) and the Reynolds number (Re) (see Figure 127).

A technical report, containing the data obtained in the investigation, was submitted to the design institute, for use in the design of the Sanmenhsia Hydro Development.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR THE STUDY OF THE THERMAL AND ICE REGIME OF WATER BODIES

Head: Professor A. M. Estifeev

IMPROVED METHOD OF CALCULATING THERMAL AND ICE PHENOMENA IN RIVERS AND STORAGE RESERVOIRS DURING WINTER

Responsible for Research: A. I. Pekhovich, Candidate of Technical Sciences, Senior Research Worker

Research Team: S. M. Aleinikov, Junior Research Worker
V. K. Dobrovol'skaya, Senior Engineer

The basic aim of the work was to improve the methods of calculating the thermal and ice phenomena and in particular to study freezing processes under conditions of wind-driven waves, as well as to measure the temperature of the water in the discharged tailwater.

The theoretical and laboratory work yielded the following results:

1. An experimental laboratory unit was designed and built for the study of ice formation in the presence of wind-driven waves. The unit also permits the study of thermal and ice phenomena, involving the use of a large amount of undercooled water and submerged ice (frazil-ice obstruction at the water intake, freezing of the trash racks at the water intake, physics of ice formation within the body of the river, etc.).

2. It was found that the freezing process of a water surface in the presence of wind-driven waves should be divided into five stages:

a) formation of ice within body of water;

b) formation of frazil-ice sheets;

c) formation of frazil-ice disks;

d) formation of ice disks;

e) freezing of space between ice disks, and formation of a continuous ice cover.

3. Tentative, empirical relationships were determined at the laboratory, expressing the relationship of the durations of the various ice-formation stages, the amount of frazil ice and the thickness of the frazil-ice sheet, to the intensity of heat transfer and the geometric parameter of the waves. It was found, i. a., that at $m\sqrt{L}$ 0.05 (m = slope of wave, L = wave length), freezing occurs almost without undercooling of the river water, and hence without formation of frazil ice.

4. A method was devised for calculating the retardation of ice formation due to wind-driven waves, and energy equations for wind-driven waves were derived for the case of frazil ice.

5. Curves were plotted permitting the determination of the hydraulics of a streamflow under conditions of a two-dimensional problem in the water-intake area, in relation to the water discharge, the depth of the water body, and the dimensions and location of intake inlets; a method was also worked out for the measurement of the temperature of the water discharged into the tailwater section.

SCIENTIFIC RESEARCH INSTITUTE OF THE GIDROPROEKT IMENI S. Ya. ZHUK

Head: I. A. Kuz'min, Candidate of Technical Sciences

INVESTIGATIONS OF THE LAYOUT OF THE SARATOV HYDRO DEVELOPMENT

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research Team: A. M. Prudovskii, Engineer
V. M. Lyatkher, Engineer

Two design variants of the layout were investigated.. In the first, the lock is located in the diversion channel; and in the second, the lock is located next to the water-conveying structures. The investigations were conducted on air-pressure models. For the tailwater section, a movable bed model was used, while the model for the headwater area was of the fixed-bed type, and was made of plasticine. On this model, use was made, for the first time, of the method of determining streamflow-velocity distribution by the erosion of movable material deposited on the fixed bed. The amount of air fed into the model is increased in steps. During the passage of a certain amount of air, the movable bottom material is carried away in those regions where the flow velocity in the model exceeds the admissible velocity for a given depth of the model. At the boundary between such regions, the flow velocities do not exceed the critical values.

The investigations disclosed that, in the case of the second design variant (lock next to the water-conveying structures), very severe flow conditions exist in the tailwater section. This refers particularly to the separating wall of the navigation canal. It is very difficult to ensure satisfactory navigational conditions at the entrance into this canal. In order to ensure normal conditions, the length of the separating wall of the lock canal should not be less than 1800 m.

No difficulties in navigation arise when the locks are located in a diversion canal. In such cases the flow velocity at the entrance into the canal does not exceed the critical value, and silting remains within acceptable limits. This design variant should, therefore, be preferred in the case of the Saratov Hydro Development.

In view of the fact that the water-discharge structures of the Saratov development are located in the flood plain where the depth in the headwater section is small compared with the depth of the river channel, the streamflow acquires a considerable slope where it approaches the water-discharge structures from the headwater side. The effect of the slope on the energy losses has been investigated. For trash-retaining structures with a large screen area, the energy losses were found to be relatively small.

INVESTIGATION OF LIGHTWEIGHT STRUCTURES USED IN THE SPILLWAY DAM AND POWERHOUSE OF THE SARATOV HEP

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research Team: N. V. Khalturina, Engineer
L. I. Bozhich, Engineer
V. T. Silkin, Engineer

The Gidroproekt designed new types of lightweight and low-cost hydro structures making use of precast reinforced concrete. These structures have been the object of special hydraulic investigations.

The study was carried out on 1:81.5 scale models of five dam bays. The dam was investigated for 2 hydraulic regimes: 1) uniform discharge of water through the whole dam frontage at a head of 4 m, corresponding to the maximum-flood passage; 2) nonuniform water passage (one bay was completely open, the other only half) and at a correspondingly greater head.

Two types of spillways were investigated: 1) trapezoidal cross section and 2) Creager profile with a vertical downstream face (overflow wall). The trapezoidal-type spillway was equipped with segment (Taintor) gates. In order to save concrete, the dam piers were sectioned with a 11.5 m clearance between sections. Such a clearance, without affecting the spillway discharge capacity, permits diversion of the streamflow with the neighboring bays when adjacent gates are opened to a different degree. On the other hand, this considerably worsens the energy-dissipating conditions. At a low elevation of the downstream apron, selected for the emergency case of one bay being fully open and all others closed, and assuming uniform discharge of a maximum flood, the energy-dissipating structures operate with low efficiency. Spillway dams with a vertical spilling face cause such conditions at the surface junction between the falling nappe and the tailwater, that the energy dissipators do not function at all.

As a result of these investigations, ways were found of improving the above spillway types, and particularly the energy-dissipating conditions, by increasing the apron elevation.

The 1:97 scale model for one section of the lightweight type of powerhouse was made for two design variants differing in the profile of the spillway outlet section not subjected to pressure. The section subjected to pressure was given the shape of an opening between the cones of the draft tubes below the floor of the turbine scroll. From the pressure openings the water reaches the cover of the draft tube and (in the first design variant) flows over the spillway overflow wall provided with double sliding gates; the second design does not provide for a spillway overfall, the spillway openings being closed by segment gates. Either variant ensures the rated water discharge as provided in the design. The main weakness of this type of spillway lies in the fact that the junction between the head and tailwater falls within the limits of the structures, and this is further aggravated by the poor streamflow and asymmetric profile of the pressure section of the water passages.

The laboratory recommends adopting a smoother profile of the water passages and increasing the length of their pressure section. These suggestions have been taken into account in the design of such structures.

WATER DISCHARGE THROUGH THE VOLGA RIVER CHANNEL RESTRICTED BY COFFERDAMS DURING THE CONSTRUCTION OF THE SARATOV POWER PLANT

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research by: A. M. Prudovskii, Engineer
V. M. Lyatkher, Engineer

The aim of this study was to check and improve the design of the head of the longitudinal temporary cofferdam and to investigate scouring and navigation conditions in the river channel restricted by such cofferdams.

The tests were carried out on an air-pressure model of the whole cofferdam and on partial models while subjected and while not subjected to hydraulic pressure.

The studies confirmed the suitability of the design for the cofferdam head, which markedly differs from the conventional design. The cofferdams of the Saratov HEP, erected on a thick alluvial deposit of fine-grained sand, are built of earth without any major slope protection.

The stability of the longitudinal cofferdam is ensured by its head, erected in the upstream corner of the cofferdam, which causes the formation of an eddy zone with low flow velocities along the whole front of the cofferdam. Usually such a head is made of sheet-pile cells. In this case, however, the upstream corner of the cofferdam is provided with a riprap-covered earth cutoff projecting some distance into the river bed. To prevent undermining of the cofferdam head, the cutoff, at its extremity close to the surface of the alluvial stratum, is covered over with rock fill, which, under the action of the streamflow, slides around the head thus forming a natural lining on the cutoff slope and protecting it from scour. This "self-lining" occurs in layers having a thickness of 2 to 3 stone diameters.

This novel design of cofferdam heads markedly reduces its cost, which is about 7 million rubles less that the cost of similar structures in other hydro developments.

INVESTIGATION OF THE HYDRAULIC REGIME AT THE TAILWATER SECTION OF THE VOLGA HEP IMENI V. I. LENIN, IN CONNECTION WITH THE RIVER-BOTTOM EROSION DURING THE HIGH WATER IN 1957

Responsible for Research: A. D. Khalturin, Engineer

Research by: I. M. Chekunaev, Engineer

The distribution of the high-water discharge of 1957 through the Volga HEP along the spillway dam was very nonuniform. Twelve turbine units operated at the right side of the dam and six bottom sluiceways were working at full opening at the turbine units Nos. 4, 5, and 6.

Downstream, in front of the turbine units Nos. 4, 5, and 6, the streamflow produced a scouring funnel whose depth below the bucket floor reached the elevation 16 m.

It was found by tests that, during the high water of 1957, the water discharge was concentrated mainly in front of the open bottom outlets where the specific discharge more than doubled as compared with the discharge at the upper head of the apron.

At an initial profile of the tailwater channel corresponding to its condition at the beginning of high water (minimum elevation + 3 m), the specific water discharge at the downstream end of the apron reached 98 m^3/sec as against 37 m^3/sec at its beginning. The maximum specific discharges increase with the scouring-funnel depth: at a funnel depth of 3 and 9 m, up to 106 m^3/sec, at a depth of 15 m, up to 113 m^3/sec.

Such a concentrated discharge in front of the six open bottom outlets at the left side of the dam in a river bed assumed to be eroded down to the elevation + 6 m, produces a hydraulic pattern similar to that noticed at the right side of the dam, where a scouring funnel was formed. The conclusion may be drawn that the hydraulic regime in the tailwater section is mainly determined by the operating condition of the water outlets of the power plant, whereas the topography of the tailwater channel plays only a minor role.

The staff also prepared an evaluation of the hydraulic losses in the tailwater channel caused by an insufficient leveling of the channel bottom.

THE BEHAVIOR OF THE TAILWATER PROTECTIVE STRUCTURES OF THE VOLGA HEP SPILLWAY DAMS

Responsible for Research: I. A. Kuz'min, Candidate of Technical Sciences

Research Team: A. D. Khalturin, Engineer
V. V. Ermakov, Engineer

This study was part of a major investigation on the behavior of tailwater protective structures of spillway dams at the Tsimlyanskii, Kakhovka, Gor'kii, Volga, and other power plants.

The report presents data on the geological structure of the spillway foundations, the sizes and types of tailwater protection, the methods of measuring the scour depth downstream from the aprons, as well as field observation of high-water discharge through the water outlets.

As a result of these observations, the following conclusions have been drawn:

1. The degree of uniformity of the water discharge along the spillway front has a direct bearing on the extent of the erosion downstream from the spillway, since a nonuniform discharge causes a flashy, shooting streamflow and hence an almost twofold increase in the specific water discharge at the scouring funnel.

2. The energy-dissipating scheme used at the spillway proved to be efficient: it ensures good conditions for the widening of the streamflow in plan, thus permitting the dam to be operated under much more severe conditions than those provided for in the design.

3. The end section of the apron designed as a bucket also proved to be efficient. Even during the emergency that arose downstream from the powerhouse, the deep scouring of the apron did not lead to the destruction of its horizontal section, and the damage stopped at a distance of 20 to 25 m from the beginning of the protecting slope of the bucket.

4. Comparison of laboratory results with field-investigation data confirmed the correctness of the method used by the Gidroproekt for the preliminary evaluation of scouring depth at different design stages.

The field investigations also showed that in selecting the energy-dissipating system, care should be taken to ensure sufficient widening of the streamflow in plan when the spillway dam is in partial operation.

HYDRAULIC CONDITIONS FOR THE DAMMING UP OF THE VOLGA RIVER DURING CONSTRUCTION OF THE STALINGRAD HEP

Responsible for Research: R. S. Gal'perin, Engineer

Research Team: A. A. Tyshkevich, Engineer
Yu. M. Skolkov, Engineer
G. N. Tsedrov, Engineer

The purpose of this study was to establish criteria for the selection of the type and the amount of materials for the construction of the dams. The study also established the most suitable sequence of work, based on the hydraulic regime of the Volga River during the damming operation (October-November 1958), and taking into account the operating schedule of the Volga HEP imeni Lenin (for water discharges varying from 4000 to 8000 m^3/sec).

The tests were carried out in two stages in a flume on a two-dimensional 1:25 scale model; firstly under conditions of a steady-flow regime with a theoretically determined, staged schedule of head increases at the embankment, and then, under conditions of a lateral outlet calculated for field conditions, i. e. for a nonsteady-flow regime.

The model tests were carried out for two design variants of the dike:

Scheme "a" — the dike is made entirely of 10-ton tetrahedral concretes blocks.

Scheme "b" — first of all a retaining mound of rock fill is erected along the whole width of the water passage opening [in the cofferdam] (a length of 300 m) to a height at which it can keep its triangular cross section; upon completion of the mound, the dike is built of of concrete tetrahedrons until its crest emerges above the surface of the water.

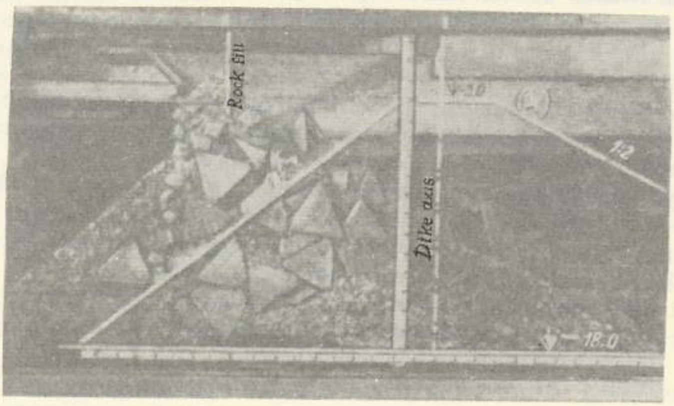


FIGURE 128. Construction of dam by filling the water passage along its whole length, to a height at which the stone mound retains its triangular shape

As a result of the model tests, the laboratory recommends the use of Scheme "b", as it requires a considerably smaller amount of tetrahedral blocks (Figure 128) The retaining stone mound prevents the blocks from being carried away into the tailwater, it scarcely affects the seepage through the dike, and it reduces the requirements for the facing on the upstream slope of the dike upon completion of the damming operations.

On the basis of laboratory experiments the following sequence of work should be adopted: for the retaining stone mound, a rock fill made of 10 to 15 cm stones, and with a volume of 50 m^3 per running meter of the prism is prepared; then the tetrahedral blocks are added up to the moment the dike crest emerges from the surface of the water. For a discharge of 4000 m^3/sec, 10 tetrahedral blocks per meter length of dam are used. At a discharge of 6000 m^3/sec, the number of blocks increases to 16; at a discharge of 8000 m^3/sec, to 20.

The upstream face of the dike is lined with concrete cubes of a uniform type, each weighing 1 t. Their number is given below:

Discharge, m^3/sec	No. of concrete cubes/m of dams
4000	28
6000	32
8000	40

The work sequence as suggested by the laboratory proved itself in practice: field investigations showed a complete agreement on the amount of material used for river damming with the amount established at the laboratory.

HYDRAULIC INVESTIGATIONS OF THE LAYOUT OF THE VOTKINSK HEP EQUIPPED WITH 10 TURBINE UNITS, AND ASSESSMENT OF SCOURING IN THE TAILWATER

Responsible for Research: D. P. Kushchin

Research by: I. M. Chekunaev, Engineer

In view of the increase in the number of turbine units of the HEP from 6 to 10 and the corresponding decrease in the number of dam bays from 11 to 8, the tests had to be repeated.

The hydro development was simulated on a 1:100 scale model.

The tests confirmed the earlier adapted outline and dimension of the connecting walls between the powerhouse and the dam and, in certain cases, showed the possibility of reducing these dimensions.

Model tests were carried out on the following types of energy dissipators:

a) pyramidal (of the Kuibyshev HEP design) with a 1:1 slope of the frontal face;

b) pyramidal with a 1:0.5 slope of the frontal face;

c) rectangular chute blocks spaced normal to the spillway-dam length;

d) wedge-type.

The effectiveness of these types was checked by arranging them in 2 to 3 rows with a different spacing.

Best flow spreading [and hence energy dissipation] were ensured by chute blocks which are more effective in reducing specific discharges and scouring at the tailwater, than are pyramidal or wedge-type dissipators.

The study estimates the extent of scouring at the tailwater channel and the volume of the rock fill in the bucket necessary for preventing scouring of the apron. It has been shown that, for pyramid-shaped dissipators, the minimum initial height of gate opening prior to complete opening may be 4.0 m; for chute blocks, 2.5 m.

INVESTIGATION OF THE TAILWATER OF THE TSIMLYANSKII HYDRO DEVELOPMENT, IN CONNECTION WITH THE EXTENSION OF THE HEP

Responsible for Research: R. S. Gal'perin, Engineer

Research by: A. I. Sal'nikov, Engineer

The extension of the existing Tsimlyanskii HEP will be carried out by erecting the powerhouse of the second stage HEP on the downstream apron of the spillway dam, and concurrently, by reducing the number of dam bays from 24 to 20. The HEP is located on erodible soil; therefore, the reduction in the length of the spillway dam will involve an increase in the specific discharges and flow velocities at the tailwater, and hence increased erosion of the channel as compared with the scouring anticipated in the initial design.

For the selection of the most suitable layout of the new powerhouse designed to ensure reduction of scouring to a minimum, a series of tests on a 1:150 scale model have been carried out; the model included the whole spillway dam and the existing powerhouse, whereas the right flood plain and part of the left flood plain lay beyond the model boundaries. The tailwater channel over a length of about 1 km was also modeled.

The tests were carried out on lined-bed models with deepened cross section assuming erosion of the river bed down to the noneroding rocks (marls) and on movable bed models in which the erodible material consisted of steamed timber sawdust. The model was checked by comparing the field velocity-distribution graphs with the graph obtained on the model of the existing hydro development.

The tests involved the study of four design variants of the new HEP to be located left and right of the dam.

As a result of the investigations, the following conclusions were drawn:

1. The location of the new HEP in the bays of the spillway dam impairs the hydraulic regime in the tailwater section by markedly increasing the specific discharge at the end of the apron, and the dimensions of the tailrace downstream from the apron.

2. All of the four design variants for the new HEP require the same protection measures against scouring at the end of the apron.

3. A certain improvement of the hydraulic regime may be attained if the design is chosen in which the powerhouse is located within the spillway dam, or by locating the turbine sets in each second bay starting from the right dam abutment.

4. For the redesign of the HEP it has been found possible to dispense with additional protective linings; however, should the erosion process during operation extend to the nearby interlayer at the end of the apron, it would be advisable to arrange a riprap filling of 10 to 30 cm stone boulders placed in a layer of 3 to 4 m over a length of 40 to 45 m.

INVESTIGATION OF A HEP WATER INTAKE WITH LATERAL OVERFLOW

Responsible for Research: R. S. Gal'perin, Engineer

Research by: A. A. Tyshkevich, Engineer

In water intakes of existing hydro power plants, the clogging of trash racks causes large losses of head. Considerable power losses are liable to occur due to the stoppage of the turbines during the cleaning of the trash racks. The discharge of water fed to the turbine units sharply increases with the increase in turbine diameter and capacity. For large power plants the power losses connected with turbine stoppage during trash-rack cleaning amount to hundreds of millions of kwhr.

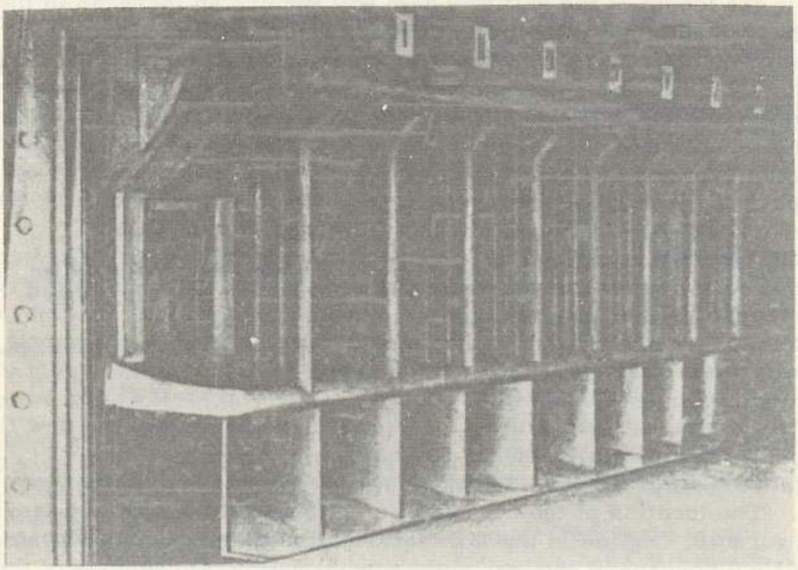


FIGURE 129. New design of HEP water intakes

In order to eliminate these shortcomings, the Gidroproekt developed new water-intake designs (see Figure 129).

In one of these designs, during the cleaning of the trash racks and lowering of the stoplog gates, the water stream is fed to the turbine units through openings in the piers. In case of partial clogging, the head losses may be

reduced by locating the racks closer to the headwater section. This involves an increase in the rack area and reduction of the flow velocity through the racks.

The laboratory tested this design on a 1:55 scale model, which included four turbine sets.

This scheme of water feed was found to ensure an almost uniform flow-velocity distribution across the stream depth (at velocities of about 1.1 m/sec) and permits the racks to be cleaned without stopping the turbine units.

Experiments show that the spacing between the trash-rack piers on the axes of the turbine scrolls should be such that the distance between the downstream edge of the first row from the upstream edge of the second row, is not less than the width of the scroll inlet. Otherwise, additional head losses are caused.

As a result of these investigations the laboratory could establish a certain sequence in the closing of the dam-inlet bays by stoplogs during trash-rack cleaning without having to stop the turbine units. Such a sequence permits the flow velocity in front of the racks to be kept below 0.3 m/sec. At the same time, the investigations showed that certain schemes of closing the bays by stoplogs were impracticable.

Experiments established a decrease in the discharge of water fed to the spiral casings of the turbine units partly covered by stoplogs.

This decrease amounted (for three closed rack openings) to:

a) 11.5% for partly closed inlets into the turbine units;

b) 4% for units adjacent to the above.

According to a tentative calculation, the annual increase in power output of the HEP with lateral-overflow intake for conditions prevailing at the Votkinsk HEP, amounts to about 56 millions kwhr or 2.8 million rubles.

FIELD INVESTIGATIONS OF LOCAL SCOURING AT THE TAILWATER SECTION OF WATER-OUTLET STRUCTURES IN HYDRO DEVELOPMENTS

Research by: I. A. Kuz'min, Candidate of Technical Science.

The study presents and analyzes data on local scouring at the tailwater side of the Kama, Gor'kii, and Volga imeni Lenin HEPs. These data served to evaluate the forecast of the local erosion depths in the tailwater section of the spillway structures. As a result of this study the following conclusions were drawn:

1. Deep scouring of the river bottom in the tailwater section of the above HEPs is the result of the markedly nonuniform opening of the spillway outlets due to the construction conditions.

2. The method (both experimental and theoretical) used by the Gidroproekt for forecasting depth of scouring is basically sound.

3. In using K. I Rossinskii's formula for determining the specific discharge in the scouring pit, taking into account the flashy character of the streamflow, allowance should be made for the effect of the piers on the nonuniform distribution of the specific discharge, at least until the specific discharge in the scouring pit is doubled. The effect of the piers on the nonuniform flow in the case of a flashy streamflow should be studied separately.

4. When the variation of nonerosive velocity from one depth to another is expressed by means of the formula $\frac{v_1}{v_2} = \left(\frac{h_1}{h_2}\right)^m$ the exponent m should not be taken as greater than 0.2.

INVESTIGATION OF HYDRAULIC CONDITIONS WHEN SHIPS ENTER LOCKS, AT INCREASED SPEED

Responsible for Research: A. D. Khalturin, Engineer

Research by: V. V. Obvival'nev, Engineer

In modern navigation locks, the water level in the lock chamber can be raised or lowered in as little as 8 to 10 minutes. However, the total lockage time still reaches 30 to 40 minutes. Moving the ship in and out of the lock chamber, positioning, and mooring still takes a long time.

Experiments were carried out to find ways of reducing the lockage time thus increasing the ship-handling capacity of the locks, without prejudicing the ships awaiting their turn in the lock channel.

These experiments were conducted on a 1:40 scale model of a 30×300 m lock chamber of the Volga type, including the 1.6 km long lower-reach lock channel with the mooring structures. The tests were made for ships of 13,300 and 5500 tons displacement and their object was to evaluate the forces acting on a ship waiting in the lock channel when another ship is passing the lock. The experiments were conducted for three ship speeds: 1.0, 1.5, and 2.0 m/sec, two ship drafts: 2.5 and 3.5 m, and four different channel depths: 5.0, 6.0, 8.0, and 10.0 m.

The investigations showed that the speed of ships of 13,300 and 5500 tons may be increased to 2 m/sec, whereby the forces acting on a 5500-ton ship waiting in the lock channel do not exceed the admissible values. The longitudinal component of the forces was 2.8 ton (admissible 3.66 ton) during the passage of a 13,300 ton, 3.5 m draft ship at a speed of 2.0 m/sec, and a water depth of 5.0 m.

The number of lockages per 24 hours can be increased by 21 by increasing the speed of the ships from 0.3 to 2.0 m/sec, assuming the length of the average path through the lock to be 500 m.

The resistance of water to the passage of a 13,300-ton ship with a draft of 3.5 m at a speed of 2.0 m/sec, and a canal depth of 5 m, amounts to 20.0 ton. For a 5500-ton ship under the same conditions, it would amount to 5.0 ton.

The water resistance increases the moment the ship enters the chamber: for a 13,300-ton ship the resistance increases from 20 to 85 ton and for a 5500-ton ship, from 5 to 20 ton, i. e., as much as fourfold.

The range of water-level fluctuations in the lock channel during the passage of ships increases with the decrease in the ratio of the water cross section in the channel to the area of the midship cross section (ω/ω_x) and the increase in the ship speed. Under test conditions the amplitude of these fluctuations is inversely proportional to the ratio ω/ω_x and directly proportional to the speed raised to a power of over 2.

THE SPILLWAY STRUCTURES OF THE VERKHNE-URAL'SK STORAGE RESERVOIR

Responsible for Research: R. S. Gal'perin, Engineer

Research by: A. P. Sal'nikov, Engineer

The spillway is located close to the left river bank where there are outcrops of rock. The water intake consists of a three-bay concrete overflow spillway and inclined outlet chute cut into the rock. According to the design, the chute has a slope of 9 m over a length of 72 m.

The aim of the laboratory tests was to check the discharge capacity of the spillway and the streamflow junction with the tailwater section and to design protective linings for the downstream face of the earth-fill dam.

The tests were carried out on a 1:60 scale model of the spillway section of the earth-fill dam.

The investigations showed the following:

1. The value of the discharge coefficient of 0.47 turned out to be higher than the design coefficient of 0.45. This made it possible to replace the concrete spillway by a spillway of polygonal cross section cut into the rock and provided with a concrete facing. The new discharge coefficient was 0.43.
2. It is recommended that the tailwater channel be gradually narrowed, while its bottom slope is decreased, thus reducing the amount of excavation work.
3. The end portion of the chute should be shaped as an energy dissipator, with a number of rock projections, which widen the streamflow, increase the energy dissipation, further reduce the amount of excavated rock, and lessen the erosion downstream from the chute.
4. Construction of a protective, water-deflecting dike built of excavated rock is recommended. This dike will reduce the flow velocity at the downstream slope of the earth dam.

INVESTIGATION OF THE WATER CIRCULATION IN THE COOLING POND OF THE ZMIEV THERMAL-ELECTRIC POWER PLANT

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research Team: V. M. Lyatkher, Engineer
S. P. Bozhich, Engineer
A. M. Prudovskii, Engineer

After studies on hydraulic and air-pressure models, the research team devised a new method for evaluating the operational efficiency of cooling ponds. According to this method the operating efficiency of the cooling pond is expressed by the degree of relative heating of the water in the pond:

$$\eta_0 = \frac{T_0 - T_{\text{nat}}}{(T_0)_{\text{sim}} - T_{\text{nat}}}, \qquad (1)$$

where T_0 = water temperature in the water outlet;

T_{nat} = natural temperature of water in the pond under given meteorological conditions, i. e., the pond temperature in the absence of waste heat water;

$(T_0)_{\text{sim}}$ = temperature of water in the water outlet at the same environmental conditions and the same temperature drop $\Delta T = T_0 - T_2$,

where T_2 = water temperature at the outlet but under "ideal" conditions of water circulation in the pond.

The "ideal" circulation pattern represents the optimum circulation conditions, i. e., a uniform utilization of the pond surface for water cooling.

The reciprocal of the relative heating $\eta_T = \frac{1}{\eta_0}$ is called the thermal efficiency of the pond, which characterizes the effectiveness of the scheme adopted for the circulation of the streamflow.

For shallow cooling ponds where the temperature can be assumed constant over the pond depth, the thermal efficiency can be determined directly from experimental data obtained from hydraulic or air-pressure models. For this purpose, one usually marks, by some method, a small volume of water entering the pond and records at its exit how long the separate portions of this volume have stayed in the pond. The time a volume of water which flows into the pond at a certain temperature remains in the pond determines its temperature at the pond outflow; hence, knowing how long all particles of the test volume remain in the pond, we can determine its temperature at the outlet and hence, the thermal efficiency of the pond.

Knowing the variation, with time $[c(t)]$ of the concentration of the test volume, the thermal efficiency of the pond can be calculated from the formula

$$\eta_T = \frac{1 - \dfrac{\int_0^\infty e^{-k\frac{\Omega}{Q} t(t_{\text{id}})} c\, dt}{\int_0^\infty c\, dt}}{1 - e^{-k\frac{\Omega}{Q}}}, \qquad (2)$$

where Ω = area of the water surface of the pond;

Q = discharge of water into the pond;

t = actual time between inflow and outflow of the test volume of water;

t_{id} = ideal time water remains in the pond equal to $\frac{w}{Q}$;

w = pond capacity (volume);

c = concentration of marked volume at the water intake at time t, as a fraction of the water discharge at the intake;

k = coefficient allowing for meteorological conditions of water cooling, $k = \frac{B_1}{\gamma\rho}$;

γ = density of water, kg/m^3;

ρ = specific heat of water

$B_1 \approx aA + B$;

A = coefficient of heat transfer by evaporation, in kcal/m^2/24 hrs. mm Hg.

a = pressure coefficient of saturated steam $a \approx 1.175 + 0.0234 \times \times (2T + \Delta T)$ mm Hg/°C;

T = characteristic pond temperature;

ΔT = variation range of temperatures in pond and in atmosphere above it;

B = coefficient of convective heat transfer (in the atmosphere), kcal/m^2/24 hrs. °C.

If the thermal efficiency of the cooling period is known, conventional calculation methods may be employed, introducing the well-known concept of "active (cooling) surface of the pond" connected with the thermal efficiency by the relationship:

$$\Omega_{act} = -\frac{Q}{k}\ln\left[1 - \eta_T\left(1 - l^{-k\frac{\Omega}{Q}}\right)\right]. \tag{3}$$

What is most important in determining the thermal pond efficiency on test models, is the determination of the time-dependence of the concentration of the marked test volume in the discharge of the water intake, $[c(t)]$. This study gives general methods and techniques for recording the marked test volume and determining function $c(t)$, both on hydraulic and air pressure models.

The validity of these formulas and of their underlying assumptions has been checked by testing warm water on a hydraulic model of the Zmiev thermal-power-plant cooling pond and on a 1.2×15 m flume.

For the conditions prevailing at the ZMIEV cooling pond, having an average depth of 4.3 m, a width of 2.85 km, and a length of about 5.8 km, the tests for determining function $c(t)$ gave results that are in good agreement with the data of thermal tests.

The new method of evaluating the cooling capacity of ponds dispenses with subjective factors inherent in the conventional method of evaluating the cooling capacity of ponds by their "active surface", and makes it possible to evaluate the characteristics of such circulation systems in which the concept of "active surface" does not apply. This method reveals new reserves in the cooling capacity of ponds which were not taken into account by the conventional methods.

The method has been used in practice at the Zmiev thermal plant to determine the cooling capacity of the cooling ponds.

The investigations were mainly concerned with the different schemes of planning a closed-cycle circulation system, with the water inlet in the center of the pond, and the outlet at its banks, inclined to the water line. The use of dikes to guide the circulating flow in the pond was also investigated. It was found that the thermal efficiency of the cooling pond for the case of a closed-cycle circulation and a single water outlet, does not exceed 0.70.

With the use of water-guiding dikes having a total length of 2000 m, the thermal efficiency of the pond increased to 0.86. If the thermal efficiency (η_T) for the same scheme is determined with the aid of the horizontal flow pattern, and using the "active zone" method, the figure 0.82 is obtained.

For all water-circulation schemes, the efficiency of the cooling pond can be markedly improved by constructing pumping stations within the pond, which direct part of the warm-water stream from the intermediate zone

into more stagnant areas of the pond. At the Zmiev pond, a pumping station having a capacity of 25% of the circulating-water discharge, replaces a water-guiding dike of 1000 to 1200 m length.

The study describes test schemes and apparatus required for the implementation of these recommendations.

INVESTIGATION OF CONDITIONS OF THE WATER INTAKE FROM THE TURA RIVER FOR THE TYUMEN' THERMAL-ELECTRIC PLANT

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research by: A. M. Prudovskii, Engineer

The purpose of this study was to determine the possibility of waste warm water, discharged into the river, being redrawn into the water intake by intake volumes; to determine the degree of silting of the water intake and the water-inlet channel; and, finally, to devise means of ensuring minimum entrainment of silt into the steam condensors.

It was found experimentally that, if the intake of water from the river is equal to, or less than, the river discharge, no entrainment of waste water occurs. In such cases the water intake is fed exclusively by fresh water. When water is drawn in at volumes greater than the river discharge, the water intake is fed by mixed water, which includes all of the fresh river water.

Tests and calculations showed that a water-intake basin in the river bed is not very effective. Its function may be taken over by the first section of the water-inlet channel, as all fractions of the sediment liable to endanger the turbine condensers will settle in the channel.

The research staff also examined the navigation conditions in the water-intake area and devised means to ensure normal navigation conditions and river-bank stability in this area.

The tests were carried out both on a rigid (nonerodible) air-pressure model reproducing a long stretch of the river, and on a hydraulic-pressure model which reproduced the intake area. The results on both models showed good agreement.

THE WATER INTAKE OF A STANDARD PUMPING STATION PROVIDED WITH SCREEN STRAINERS

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research by: N. V. Khalturina, Engineer

The water intake of a standard pumping station was investigated on a 1:11 scale model. The intake is an integral part of the pumping station, equipped with OP-2-145 and OP-5-145 pumps, which supplies water to the

thermal power station. It is located either on the bank of the source of the water supply, or on the water-conveying channel. The chambers of the water intake are equipped with rotary screens, which may be installed normal to the water flow, or parallel to the chamber walls. In the latter case, water is fed to the screens, flowing from the walls toward the chamber axis, and it reaches the suction pipe of the pump from the space between the screens, passing through the clean-water chamber. Such a screen arrangement has its advantages from the point of view of water purification; however, in the case of an unfavorable layout of the screens with respect to the chamber axes and the suction pipe inlet, the flow condition at this inlet may be unsatisfactory.

The aim of the investigations was to arrive at a suitable screen layout and to improve the flow conditions at the suction-pipe inlet.

The tests were carried out for 4 screen spacings and 3 different distances between the screens and the pipe, and involved the recording of velocity and pressure distribution in the inlet cross section of the pipe, and of the level fluctuation at the intake, as well as the study of water flow by means of dyes. These tests permitted the evaluation of nonuniform distribution of flow velocities at the pipe inlet, the losses in the water intake and the assessment of funnel formation in the clean-water chamber.

In all the cases investigated, flow funnels were observed in the clean-water chambers and this may lead to air entrainment into the suction pipe.

At a small width (0.7 to 0.75 of the suction-pipe diameter) of the inter-screen space, the flow-velocity distribution at the suction pipe is very nonuniform, the stream filaments being forced away from the walls. By increasing this width to 0.83 of the pipe diameter, this nonunnformity is somewhat, but not sufficiently, reduced.

In order to avoid funnel formation and to ensure uniform flow across the channel width, the most effective means is to install screens in the clean-water chamber, both in a vertical and a horizontal plane.

At a recess depth not exceeding 0.2 to 0.3 m, the screens in the case of a frontal inlet ensure uniform inlet flow without funnel formation. The increase in width of the recesses causes the appearance of funnels and disturbs the uniform water inlet flow.

The study has been used in the design of standard pumping stations by the Teploelektroproekt.

HYDRAULIC INVESTIGATIONS OF CHECK VALVES

Responsible for Research: Professor S. A. Egorov, Doctor of Technical Sciences

Research Team: N. V. Khalturina, Engineer
L. I. Bozhich, Engineer.

The purpose of the study was to determine the resistance coefficient and streamflow pressure in a check valve installed at the pressure pipe of the pumping station of a thermal power plant. The object of the check valve is to close the pipe automatically in case of unforeseen stoppage of the pump and formation of a return flow.

Design parameters were supplied by the contractor and had not been modified.

At a given shape of the valve, the resistance coefficient, pressure distribution on the valve surface and body, and streamflow pressure on the valve body, were found to depend largely on the Reynolds number. Therefore, no model studies could be made on the basis of the Froude number; the investigations were carried out for a large range of velocity variations assuming as calculation parameters corresponding to field conditions the magnitudes falling in the "self-modeling" region.

For a direct (forward) flow, the studies yielded a resistance coefficient of 0.22; for a return flow, 0.5.

The increase in flow velocity at the inlet into the valve was caused by the marked decrease of pressure at the pressure side of the valve, a fact which led to spontaneous opening of the valve against the flow direction in case of forward flow, when pressure was applied at the end of the flow area in the cavity of the valve body. The measurement of the force of pressure on the valve, by both weight and piezometric methods, showed that, at a full or partly opened valve (up to 1/4 of the full opening), the check valve will be subject to negative pressure forces (back pressure) directed opposite to the streamflow.

Of course, the most efficient means for eliminating these phenomena would be a modification of the valve design, but since the valve design could not be altered, the necessity arose of searching for other solutions. The problem was solved by reducing the pressure at the inside half of the valve body by establishing, in the area of maximum pressure decrease, a communication between the internal valve cavity and the valve surface. However, the necessary decrease of pressure can only be obtained in a sufficiently watertight valve.

TNISGEI IMENI A. V. VINTER HYDRAULIC ENGINEERING LABORATORY

Head: L. G. Gvelesiani, Candidate of Technical Sciences, Lecturer

HYDRAULIC INVESTIGATION OF TUNNELS

Responsible for Research: Yu. K. Fogel', Engineer, Research Worker

Most of the water tunnels of the U. S. S. R. are lined with gunite. The roughness coefficient of such linings has not so far been investigated. During the last 10 years the TNISGEI investigated most of these tunnels.

Figure 130 shows the results of investigations on most typical gunite-lined pressure tunnels Due to the particular conditions under which these investigations were carried out (high accuracy in measuring the geometrical parameters of the inside surfaces and the hydraulic characteristics, uniform surface, absence of joints, and wide test range), they may in fact be considered to be laboratory investigations carried out on a natural-size model.

The following parameters were investigated: a) Reynolds number over a range from 15×10^3 to 15×10^6 (Figure 130 shows the maximum Re number for tunnel No. 5 to be 10^7, for all the tunnels investigated the Re number range varied from 10^6 to 5×10^6); b) range of variation of relative smoothness

d/Δ from 1000 to 17,000. Most water conduits, not only tunnels but also pipes and canals of hydro plants, fall within these ranges of Re numbers and relative smoothness.

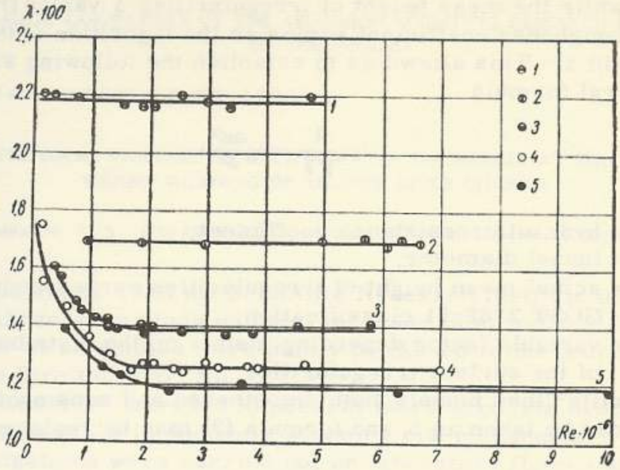


FIGURE 130. Relationship between λ and Re and $\frac{d}{\Delta}$ for gunite-lined pressure tunnels

1 - $\frac{d}{\Delta}$ = 1080; 2 - $\frac{d}{\Delta}$ = 3100; 3 - $\frac{d}{\Delta}$ = 7700; 4 - $\frac{d}{\Delta}$ = 12,600;

5 - $\frac{d}{\Delta}$ = 17,000.

For all gunite-lined nonsmoothened tunnels built in the U. S. S. R. the roughness coefficient turned out to be 0.016 ± 3%. The Technical Specifications and Norms do not give a precise value for this coefficient, but only indicate a range (0.016 to 0.023). This should be replaced by a definite value — 0.016. Smoothing with a wire brush has little influence on the wall roughness. The mean height of the surface irregularities, as established by geometrical measurements, varies from 2 to 2.5 mm. Tunnel No. 1 is typical of a nonsmoothened gunite-lined structure. Tunnel No. 2 is an intermediate type, with partly smoothened inside surfaces, and has only theoretical interest.

Tunnels Nos. 3, 4, and 5 belong to the class of smoothened gunite-lined conveying structures built after the war, which is of particular importance in hydro construction. The roughness coefficients of tunnels Nos 3, 4, and 5 is 0.0125. This relatively high value is due to poorly smoothened gunite linings, as well as to the already smoothened portions being sprayed with concrete while guniting adjacent sections. By proper smoothing, the coefficient can be reduced to 0.012.

Thus, for smoothened gunite-lined tunnels, a value of 0.012 ± 3% may be adopted for the roughness coefficient instead of a range of 0.012 to 0.015 as given by the Technical Specifications. The mean height of surface irregularities [for smoothened surfaces] obtained by geometrical measurements

in tunnels Nos 3, 4, and 5 and confirmed in other tunnels, varies from 0.2 to 0.3 mm in the areas where the smoothness was affected by one of the factors mentioned above. In other areas the height was less than 0.2 mm, while in certain sections a height of less than 0.1 mm was measured.

The roughness coefficient of internal tunnel surfaces varies from 0.01 to 0.040 while the mean height of irregularities Δ varies from 0.02 to 200 mm, i. e., the roughness coefficient varies as the logarithm of the variation in the mean height Δ. This allows us to establish the following simple formulas

A general formula

$$\frac{1}{\sqrt{\lambda}} = \log\frac{ad^2}{\Delta^2}, \tag{1}$$

where λ = hydraulic-resistance coefficient;

d = tunnel diameter;

Δ = actual mean height of irregularities corresponding to H_{mean} in the GOST 2788-51 classification;

a = variable factor depending mainly on the distribution and shape of the surface irregularities.

For gunite-lined tunnels, both smoothened and nonsmoothened the coefficient a can be taken as 5, and formula (2) may be replaced by

$$\frac{1}{\sqrt{\lambda}} = \log\frac{5d^2}{\Delta^2}. \tag{2}$$

By transforming (2) following Agroskin's method, we obtain

$$c = 17.7\log R + {}^1/_n, \tag{3}$$

$$^1/_n = 16.9 - 17.7\log\Delta, \tag{4}$$

where Δ is given in mm.

The cumbersome mathematical apparatus given in the Technical Specifications may be replaced by the simpler and more accurate formula (4). The formula may also be applied for the checking of lining quality and roughness of tunnels.

This may be done as follows:

a) For nonsmoothened gunite linings there is no need to control the roughness, but in the top gunite layer one should avoid using sand having a gunite size larger than 3 mm.

b) For smoothened gunite linings a value Δ or H_{mean} of 0.1 mm must not be exceeded. The consistency of the gunite should be such that it can be smoothened in one operation eliminating not only roughness but also waviness. Seams between adjacent smoothing operations should be avoided, and any concrete splashes on an already smoothened surface should be removed. The smoothened surface of the tunnel invert should be in line with the rest of the tunnel surface, etc.

These investigations led to the following conclusions: the more exact values for the roughness coefficients, and geometrical parameters of the inside tunnel surfaces, and the use of a single calculation formula, permit greater accuracy of hydraulic tunnel calculation and, by establishing

smaller roughness coefficients, cost and duration of tunnel construction may be reduced.

HYDRAULIC LABORATORY OF THE ALL-UNION SCIENTIFIC RESEARCH INSTITUTE OF THE VODGEO

Head: N. P. Zrelov, Candidate of Technical Sciences

ON THE LOCAL INCREASE OF SPECIFIC WATER DISCHARGES AT PLACES OF SUDDEN WIDENING OF AN OPEN RIVER CHANNEL

Responsible for Research: A. S. Obrazovskii, Candidate of Technical Sciences, Senior Research Worker

At the beginning of 1958 the Scientific Research Institute of the VODGEO completed a three-year study on local increase in specific discharges. The results of these studies were partly published in the journal "Gidrotekhnicheskoe Stroitel'stvo" No. 12. 1957.

These studies were intended to present more accurate hydraulic calculations of the streamflow in suddenly widening river channels.

The investigations were carried out on laboratory flumes and river-channel models. To exclude nonsteady flow conditions in the test flume, the studies were conducted on streamflows with unilateral sudden widening in plan and in depth $\left(\frac{B}{b} = 1.0 \text{ to } 3.0\right)$. For river-channel models the test conditions admitted a bilateral widening in plan $\left(\frac{B}{b} = 10;\ \frac{h_2}{h_1} = 1.3 \text{ to } 4\right)$. The break in the channel bottom was simulated by a vertical step on the model channel-bottom and by 1:4 and 1:8 slopes. It should be noted that the stream, forced to flow through the widened stretch of the flume, exhibited steady-flow conditions ($\alpha_0 = 1.03$ to 1.07).

The tests showed that the degree of stream widening in plan has no visible effect on the coefficient of local increase of specific discharges $\frac{q_2}{q_1}$. On the other hand, the influence of a sudden break in the channel bottom is very marked, so that, even in a rectilinear stream $\left(\frac{B}{b} = 1.0\right)$, in the presence of a step on the river bottom, a flow regime may be established in which the value of coefficient q_2/q_1 will be much above unity.

On the stretch of sudden open-channel widening, the streamflow was found to have three characteristic regimes, two of which may be considered as boundary regimes, and the third, as a mixed regime. One of the boundary regimes, also termed deformation regime, is characteristic in that over the widening section both flow velocity and cross section of the flow filament are maintained at constant values. Under such a regime the coefficient of local increase of specific discharges $\frac{q_2}{q_1}$ is equal to the depth ratio, i. e., $\frac{q_2}{q_1} = \frac{h_2}{h_1}$.

The second of the boundary regimes, termed also the regime of normal widening, is characterized by an extremely intensive water exchange between the flow filament and the water space surrounding it at the widening stretch. In this case, the coefficient of local increase of specific discharges

will have the lowest value $\left(\frac{q_2}{q_1}=a\sqrt{\frac{h_2}{h_1}}, \text{ where } a\approx 0.95\right)$. The third of the above regimes, which is much more common, is an intermediate stage being characterized both by a marked water exchange of the flow filament and by deformation of its cross section.

In a series of tests in which the test conditions remained unchanged, the flow regimes obtained were nevertheless different and no criterion for transition from a regime of normal widening to a deformation regime could be established.

Streamflow widenings beyond the step on the channel bottom have almost no longitudinal pressure gradient. Stream widening of the slopes creates backwater pressure which proved to have the same value ($z_1=\frac{v_1^2}{g}\cdot 0.20$) for different tests. In the region of lateral whirls the piezometric pressure is higher than in sections of maximum contraction $z_2=0.055\,\frac{v_1^2}{g}$.

The tests also showed that the coefficient of local increase in specific discharge at channel widenings is mainly determined by the conditions prevailing at the so-called initial section of the widening stretch. In other sections of the widening stretch the nature of flow conditions has no major effect on the magnitude of the coefficient $\frac{q_2}{q_1}$.

The test results have been extended theoretically to the case of a sudden widening of a stream, flowing over a step on the channel bottom. For the slice of the flow filament taken at the portion between the step and the section of largest flow contraction in plan, the equation of impulses has been derived

$$\frac{q_2}{q_1}=\frac{\bar{\alpha}_{01}-\frac{gz_1}{v_1^2}}{\bar{\alpha}_{02}\left(1+\frac{\Delta Q}{Q}\right)}\cdot\frac{h_2}{h_1}, \tag{1}$$

where ΔQ = water-exchange quantity.

For the same slice the laboratory also derived the energy-balance equation in which allowance is made for energy losses due to formation of eddies, water exchange, and forward motion of water layers united as the result of water exchange.

From the energy-balance equation in the flow slice we obtain

$$\frac{\Delta Q}{Q}=\frac{\bar{\alpha}_1-2\frac{gz_1}{v_1^2}}{2\bar{\alpha}_2}\cdot\frac{v_1^2}{v_2^2}-0.50. \tag{2}$$

Inserting this expression in (1) we obtain

$$\frac{q_2}{q_1}=f\left(\frac{\bar{\alpha}_{01}}{\bar{\alpha}_{02}},\ \frac{\bar{\alpha}_1}{\bar{\alpha}_2},\ \frac{gz_1}{v_1^2},\ \frac{h_2}{h_1}\right).$$

For $z_1 \cong 0$ which is valid for the flow widening beyond the step, we obtain

$$\frac{q_2}{q_1}=\frac{\dfrac{\overline{\alpha}_1}{\overline{\alpha}_2}\cdot\dfrac{h_2}{h_1}}{\dfrac{\overline{\alpha}_{01}}{\overline{\alpha}_{02}}+\sqrt{\left(\dfrac{\overline{\alpha}_{01}}{\overline{\alpha}_{02}}\right)^2-\dfrac{\overline{\alpha}_1}{\overline{\alpha}_2}}}. \tag{3}$$

This relationship holds good for all three widening regimes; q_2 and q_1 in (3) are taken as the average value across the width of the stream line (flow filament).

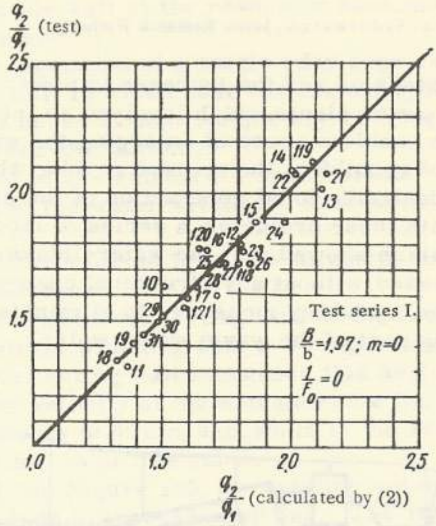


FIGURE 131. Agreement between test and calculated values of $\frac{q_2}{q_1}$

When comparing the results of calculations by this formula with the test results we assumed that

$$\overline{\alpha}_{ot}=\alpha_{oz}\,\alpha_{oy}\,\alpha_{ot},$$

where α_{ot} is a coefficient allowing for velocity fluctuations.

The correction coefficients α_{oz} and α_{oy} were determined from the graphs of velocity distribution in the horizontal and vertical plane while α_{ot} was determined by the approximate formula

$$\alpha_{ot}=1+0.11\left(\frac{1.2\,v_1}{v_2}-1\right)^2. \tag{4}$$

The relationship between the corrections for velocity fluctuations and kinetic energy is found from the well-known formula

$$\bar{\alpha}_2 = 3\bar{\alpha}_{02} - 2. \qquad (5)$$

Under these conditions the similarity between test and calculated data proved satisfactory, as may be seen from Figure 131.

IMPROVING THE SHAPE OF THE INLET INTO THE WATER-INTAKE BASIN

Responsible for Research: A. S. Obrazovskii, Senior Research Worker

Research by: G. B. Voino-Sidorovich, Junior Research Worker

In view of the extension and the increase in power output of certain nuclear and thermal power plants which involved an increase in the water-intake capacity, the problem arose of enlarging the water-intake basin by as much as 50%, and of rebuilding the protective dike, which, however, would have caused an undesirable local contraction of the streamflow.

In connection with these problems, a series of laboratory investigations were carried out which showed that the water-intake capacity of the reservoir could be increased, without any structural changes, by increasing its depth by about 1.5 m, by taking measures to eliminate excessive reservoir silting, and by using part of the warm waste water.

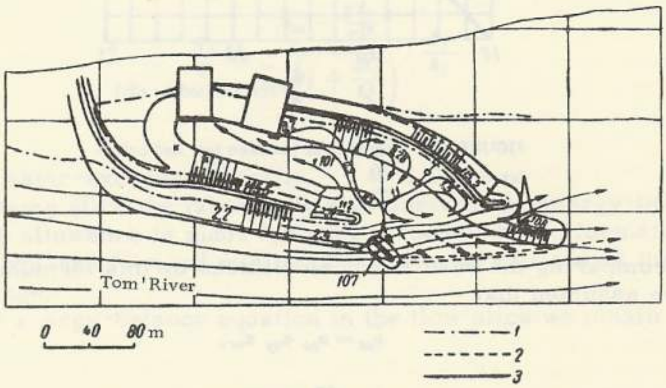


FIGURE 132. Improved designs of intake-reservoir inlets

1 - shore line of reservoir; 2 - bottom streamflow lines; 3 - top streamflow lines.

Figure 132 shows some improved designs of inlets into the water-intake reservoir selected as a result of model tests. These inlets have upstream and downstream dikes and a special water-deflecting wall at the downstream bank.

The upstream (spur-shaped) dike is flooded during high and flood water, but it protects the intake entrance against frazil ice and silt deposition. Figure 132 shows the water currents induced by this dike at the intake entrance during high waters. This scheme of flow regulation is of particular value in that it diverts the bed load from the eddy region at the water intake.

The downstream dike serves to protect the intake entrance against the suction of the warm water and to guide the feeding-stream lines toward the intake structure, so that during ice drift the intake reservoir will have rectilinear currents of small flow velocity.

The downstream dike may or may not be flooded during flood water. In the latter case (as shown in Figure 132) the currents of the intake entrance are steadier and this, in turn, leads to greater steadiness of flow in the reservoir itself.

The water-deflecting wall at the reservoir bank, made of prefabricated pile structures, is intended:

a) to prevent the formation of a single eddy zone extending over the whole volume of the reservoir, in which case the exchange of water between river and reservoir would cease almost completely, causing considerable silt deposition on the reservoir bottom;

b) to create at the intake entrance a small primary eddy region which prevents stagnation of the water within the reservoir, and hence, silt deposition;

c) to create conditions for an intensive bilateral widening of the stream lines within the reservoir.

In the absence of a water-deflecting wall at the reservoir bank, the stream line exhibits a unilateral widening with a divergence angle from 0°40' to 1°. The existence of a deflecting wall increases this angle to 2 - 3°. In the first case the stream-line velocity at the entrance into the water intake of the pumping station amounts to 60 cm/sec, while in the second case, the flow velocity is only 1/4 to 1/5 of this value.

As can be seen from Figure 132, the existing water-intake reservoir has a wide inlet which leads to increased silting. The improved design provides for a reduced entrance width by building a river dike. In order to prevent any additional contraction of the river stream (due to erection of this dike), the section of the dike adjoining the entrance into the intake reservoir is designed so as to ensure its flooding by flood waters of low recurrence. The overflow over the crest in this section of the dike at particularly high floodwaters does not lead to any visible disturbance of the flow regime within the reservoir.

IMPROVED METHODS OF MODELING HARBORS. INFORMATIONAL DATA ON SIMILARITY PROBLEMS

Research Team: A. S. Ofitserov, Doctor of Technical Sciences
I. A. Vaisfel'd, Candidate of Technical Sciences

During the years 1955 to 1958, the Kuchino Hydraulic Engineering Laboratory of the VODGEO Scientific Research Division carried out special scale tests on laboratory flumes and three-dimensional models with the aim of

determining correction coefficients for Froude numbers which allow for internal friction.

The conditions of a wavy stream, as well as river-channel processes, are characterized by the Re number. In a wavy stream, however, the flow velocity, taken as an average over the stream cross section, is replaced by the (surface or average) peripheral velocity, while the length element is replaced by the wave height or water depth. The correction coefficients are given as a function of the Re number and may, in small models, attain a considerable magnitude.

It is shown that the relationships and methods obtained in this study may be used for refining the existing methods of designing harbor models, and also for converting laboratory data from model to field conditions.

SCIENTIFIC RESEARCH INSTITUTE OF THE UkrVODGEO OF THE ASiA OF THE UKR. S. S. R. LABORATORY FOR HYDRO STRUCTURES

Head: A. S. Tseitlin, Candidate of Technical Sciences, Senior Research Worker

POWER-CANAL HEADWORKS OF THE EGORLYK HEP NO. 1. INVESTIGATION OF SEEPAGE FROM THE STORAGE RESERVOIR AROUND THE LEFT WING OF THE DAM

Scientific guidance: A. S. Tseitlin, Candidate of Technical Sciences, Senior Research Worker

Responsible for Research: V. A. Shilin, Acting Senior Research Worker

Research by: E. S. Troyanik, Junior Research Worker

One of the measures designed to ensure stable and reliable operation of the pressure basin and the pipelines located at the left slope of the valley is the interception and control of the by-pass seepage flow from the canal and from the storage reservoir, around the left wing of the dam.

Calculation and design of the interception structure are not easy since, without any special investigations, it is very difficult to establish accurately the parameters of the seepage flow.

The study gives the scientific foundation of the methods employed, the investigation results, conclusions, and practical recommendations.

The problem of the by-pass seepage from the storage reservoir around the left wing of the dam has been solved by plotting the flow net in plan according to a method developed by Professor S. F. Aver'yanov, while the equations for seepage in straight prismatic channels have been used for corrections to the flow net during plotting.

The area of flattening of the flow seepage from the canal has been determined by means of the relationships obtained from an approximate hydraulic consideration of the problem.

The investigations made it possible to determine the general pattern and the parameters of the by-pass seepage flow, to evaluate the effectiveness of the measures intended for intercepting and diverting the seepage flows and, on the basis of the data obtained, to recommend the necessary design and dimensions of the seepage drains.

This material was submitted to the design institute where it served as a basis for the working design.

Following the recommendations of the institute, it was possible to reduce the construction cost by 786,000 rubles.

SCIENTIFIC RESEARCH INSTITUTE OF THE UkrVODGEO OF THE ASiA
Ukr. S. S. R. LABORATORY FOR INDUSTRIAL (ENGINEERING) HYDRAULICS

Head: Professor G. A. Petrov, Doctor of Technical Sciences

PROTECTION OF SLOPES OF EARTH-FILL STRUCTURES AGAINST WAVE ACTION

Scientific guidance: Professor G. A. Petrov, Doctor of Technical Sciences

Responsible for Research: M. I. Lupinskii, Candidate of Technical Sciences, Senior Research Worker

Wind-driven waves exert a destructive action on the slopes of earth-fill structures.

For several years, this laboratory has been carrying out special research to study the mechanism of interaction between wind-driven waves and protected slopes, as a result of which certain methods for calculating and designing slope protection have been worked out.

The experimental part of these investigations was carried out on a specially designed wave-modeling flume of relatively large dimensions, equipped with a mechanical wave generator. Sizes of both flume and generator were chosen with a view to permitting investigations to be made at a wave height $2h$ = 0.42 m.

The invastigations involved the study of conditions for stable operation of stone protection (stone facing and riprap) and concrete slabs.

The methods for obtaining the quantitative parameters were based on the principle of determining the interrelation between the sizes of the structural elemems of the given slope protection (sizes of stones, slabs, and crushed stone, thickness of crushed-stone layer, stone facing, and riprap) and the sizes of waves causing destruction of the slope protection, allowance being made for all factors involved.

These investigations showed that during the action of waves on the dam slopes the following situation may occur, leading to destruction of the slope protection and, consequently, of the slope itself:

1) when the wave breaks, the mass of water which forms the wave crest exerts a dynamic effect (wave impact);

2) after the wave impact, the water mass flows up the slope and then down again, and this movement extends through the thickness of the slope protection, and also below the protective slabs (depending on the sizes and numbers of the joints).

The basic problem of proper slope protection is therefore:

1) dissipation of the wave-impact energy. This function is fulfilled by the surface of the slope protection;

2) slowing the movement of the water (after wave impact) along the slope at the contact between the slope and the bottom surface of the protective lining, so as to avoid the dislocation of individual elements of the slope protection. This is achieved by the crushed-stone layer below the outer lining.

Thus, from the point of view of constructional considerations, the facing which protects the slopes of earth-fill structures against wind-driven waves should consist of an outer lining placed over an underlying layer.

Until recently, multilayer inverted (graded) filters have been used as such underlying layers. It has been found, however, that such filters may be dispensed with by replacing them with a single layer of crushed stone of different grain size. The size grading has to be chosen so as to include grains beginning from sizes of the skeleton of the slope protection (d_{nm}) to large sizes forming the skeleton of the underlying layer itself (d_{nc}). The sizes of the latter grains d_{nc} are selected so as to prevent them from passing through the voids in the upper lining.

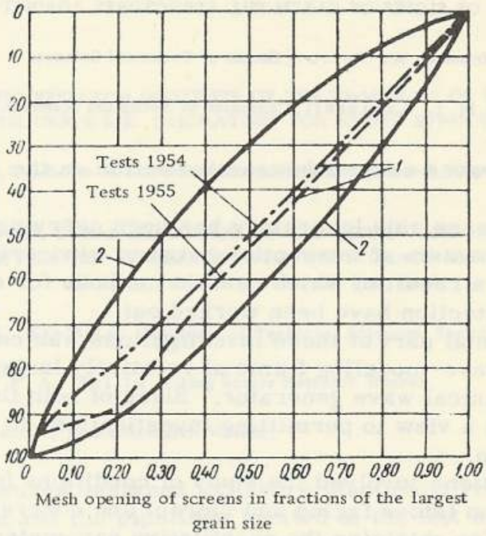


FIGURE 133. Screening curve for crushed-stone grains

1 - screening curve for d_{nc}= 50 mm; 2 - boundary of size grading recommended for crushed stone.

Over the grain-size range from d_{nm} to d_{nc}, the grain-size distribution should be chosen so that the screening curve be within the limits of the recommended grain sizes (Figure 133). The thickness of the underlying layer t_{und} should not be smaller than the critical value formed from

$$t = \sum_{i=1}^{i=n} \frac{d_{nc}}{2{,}5^{i-1}}, \tag{1}$$

where n is the number of imaginary unit layers of an ideal inverted filter considered to be built up so that particles of smaller size cannot pass through the voids between particles of larger size. The value for n may be taken within the range from 3 to 4, which is sufficiently accurate for practical purposes.

The constructional sizes for the upper slope lining may be determined from the following relationships:

A. For stone facing

$$Q_{av} = \eta \cdot 11(2h)^3, \tag{2}$$

where Q_{av} = average weight of stones kg;

$2h$ = wave height, m;

η = safety factor whose magnitude, in relation to the slope inclination is assumed to be between 1.25 and 1.50 (for a steeper slope η assumes a greater value).

Thickness of upstream lining:

$$t_y = 1.36\sqrt[3]{\frac{Q_{av}}{\gamma_{st}}}, \tag{3}$$

where γ_{st} = bulk weight of stone, kg/ m^3.

The sizes of the individual stones (boulders) may vary within

$$2Q_{min} \geqslant Q_{av} \geqslant \frac{Q_{max}}{2}. \tag{4}$$

B. Riprap

$$Q_r = \eta \frac{8.0 - 1.4\,m}{1000\beta}\ \gamma_w(2h)^3, \tag{5}$$

where m = slope coefficient;

$\beta = \frac{2h}{2L}$ = steepness of wave;

γ_w = volume weight of water, kg/ m^3;

η = safety factor taken within the range from 1.00 to 1.25 for slopes with a coefficient m of 5 and 2, respectively.

Thickness of riprap layer

$$t_r = 2\sqrt[3]{\frac{Q_r}{\gamma_{st}}}. \tag{6}$$

C. Slabs

$$t_{sl} = \eta \frac{0,11 \cdot 2h}{(\gamma_{sl} - \gamma_w)\sqrt{B}} \frac{\sqrt{m^3+1}}{m}, \tag{7}$$

where t_{sl} = slab thickness;

B = length of slab edge (square) in the horizontal plane;

γ_{sl} = bulk weight of slab;

η = safety factor assumed between 1.25 and 1.50.

Relationship (7) gives the sizes for slabs that resist displacement under wave action.

At present, protective structures are being designed and built for the earth dams of the Kakhovka, Dubossary and Kremenchug HEPs. Replacement of a single-stratum layer permitted the volume of crushed stone for the protective structures to be reduced by 65,000 m^3.

MISI IMENI V. V. KUIBYSHEV CHAIR OF HYDRAULICS

Head: Professor V. D. Zhurin, Doctor of Technical Sciences

INVESTIGATION OF NORMAL OPERATIONAL AND EMERGENCY CONDITIONS FOR THE INCLINED SHIP LIFT OF THE BRATSK HEP

Scientific Guidance: O. F. Vasil'ev, Candidate of Technical Sciences, Lecturer

Research Team: N. Kh. Gol'tsov, Engineer
Yu. N. Shubin, Engineer

The aim of this study was to carry out theoretical and experimental research on the hydrodynamic processes in the ship-carrying chamber of the inclined ship lift of the Bratsk HEP, both during normal operation and, particularly, under emergency conditions.

The flow regime in the ship-carrying chamber was investigated taking into account: the effect of the size and draft of ships on the fluctuation of the water level within the chamber; the swing of the ship and the stresses in the mooring structures. These calculations yielded the permissible braking acceleration of the chamber during emergency cases.

Tests were also carried out to study the movement of ships within the chamber when the mooring cables break due to a sudden stoppage of the chamber. The possibility of reducing the weight of the ship-carrying chamber by reducing the safe (critical) depth below the ship bottom was also investigated.

The tests were carried out on a 1:50 scale model of the ship-carrying chamber. The model calculations were based on the laws of gravitational similarity.

A special method, checked subsequently in practice, has been developed for calculating the water-level fluctuations in the ship-carrying chamber and the longitudinal forces acting on the ship.

The study may be useful in the design of ship lifts for hydro developments. At present the Moscow branch of the Gidroenergoproekt is using the results of the study in the design of the inclined ship lift of the Bratsk HEP.

MISI IMENI V. V. KUIBYSHEV CHAIR FOR WATER-POWER ENGINEERING

Head: Professor F. F. Gubin, Doctor of Technical Sciences

INVESTIGATION OF THE HUB EXTENSION OF THE KUIBYSHEV HEP TURBINES

Scientific Guidance: Professor F. F. Gubin, Doctor of Technical Sciences

Responsible for Research: M. F. Gubin, Candidate of Technical Sciences, Lecturer
V. Ya. Karelin, Candidate of Technical Sciences

This study deals mainly with the efficacy of increasing the length of the hub extension of the PL-587 adjustable-blade turbine runner installed at the Kuibyshev HEP. Near the hub extension there is a sudden widening of the

water flow (the angle of widening reaches 33-34°). This angle may be considerably reduced by lengthening the hub extension and by giving it a smoother curvature. However, these structural modifications increase the friction losses in the water and hence diminish the effect of the increased hub-extension length. The first part of this study deals with the design of the optimum shape of the hub extension. It includes the investigation of the power-engineering parameters, the effect of the hub extension on the axial hydrodynamic forces acting on the turbine, the effect of cyclic loads acting on the hub extension, and the hydrodynamics of the stream flow in the draft tube. The tests were carried out on runner models having a diameter of 180 and 250 mm. Six different shapes of hub extensions (of varying lengths) were investigated and the optimum length determined. This length turned out to be greater than that of hub extensions now used. Thus, as against the actual extension length of 1.23 times the hub diameter, the optimum length (determined experimentally) is 2.46 times the hub diameter. The average increase of the turbine efficiency (for a runner diameter of 9.3 m) amounted to 0.40-0.45% at all operational conditions. This increase is accompanied by a reduction of at least 3-8% of the hydrodynamic axial forces acting on the turbine.

Firstly, the cyclic loads acting on hub extensions of different lengths, under varying operational conditions of the turbine, were measured, and the resulting stresses in the fastening elements of the hub extension were then calculated.

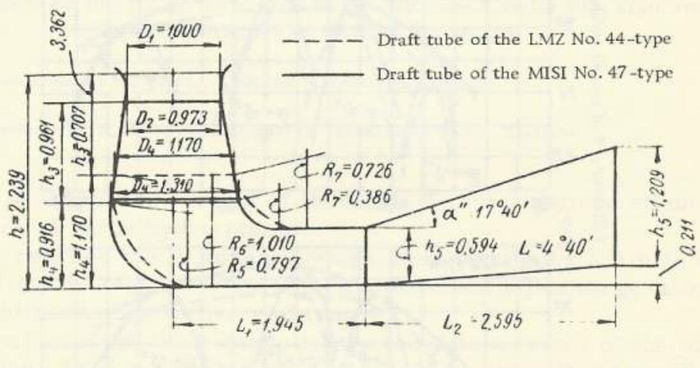


FIGURE 134. Curved draft tube designed for the Volga HEP imeni V. I. Lenin

The stresses in the fastening screws of conventional hub extensions or in those of greater length did not exceed 200 kg/ cm^2 under all operational conditions of the turbine.

The study also involved motion-picture photographs of the stream flow below the turbine wheel and in the draft-tube elbow. The motion picture shows the test unit and the flow pattern below the runner, in the first section of the draft tube, under various operational conditions of the turbine.

The experimental results were confirmed theoretically.

Special consideration was given to the suitability of using, in conjunction with the PL-587 turbine wheel, draft tubes having a relatively high cone.

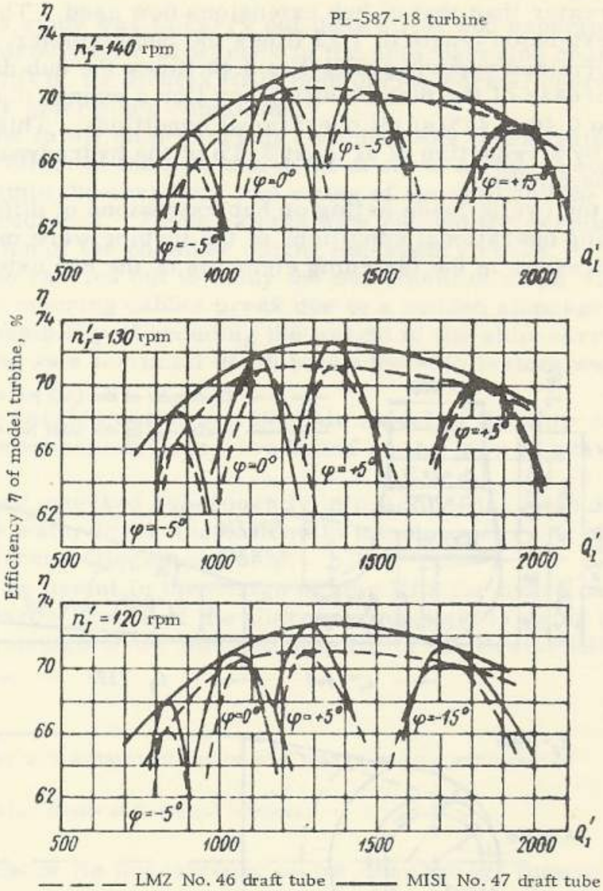


FIGURE 135. Curves showing increase in efficiency of a model PL-587-18 turbine

The tests showed that the increase in height of the draft-tube cone without changing the over-all height of the draft tube, ensures a more uniform distribution of flow velocities and pressures over the cross section of the draft tube. For a draft tube having a height of $2.24D_1$, it is practical to

880

increase the cone height up to $\sim 0.86D_1$. Some widening in the horizontal plane of the elbow of the curved draft tube also increases the discharge capacity of the turbine and hence its efficiency. It appears advisable to increase the [curvature] radius R_8 to $0.70D_4$ as compared with the usually accepted value, $0.58D_4$.

The curved draft tube with a height of $2.24D_1$ especially designed for the Volga HEP imeni I. V. Lenin (Figure 134) increased the efficiency of the PL-587-18 model turbine by an average of 1.5% for all the flow regimes considered (Figure 135). This corresponds to an increase in the output of the plant amounting to 80-100 million kwhr in an average year. The curved draft tube of the recommended design does not involve any changes in the over-all size of the turbine block (height, width, and length) and even permits a certain reduction in the volume of the concrete masonry.

The results obtained in this study are also of interest as regards the use of small-scale models in the power-engineering tests of hydraulic turbines. It was found that the head on the experimental unit materially affected the power-engineering characteristics of the model turbine (for runner diameters of 100 and 180 mm). The extent of this influence depends on the runner type, operational conditions, and the shape of the component elements of the water passages of the turbine.

INVESTIGATION OF UNSTEADY FLOW REGIMES AT THE MINGECHAUR HEP, RESULTING FROM THE INCREASE IN HEADWATER LEVELS

Scientific Guidance: Professor F. F. Gubin, Doctor of Technical Sciences

Responsible for Research: V. A. Orlov, Candidate of Technical Sciences, Lecturer

The work presents the results of theoretical investigations on unsteady flow processes at large hydro plants.

The water reaches the turbines through steel penstocks 5.3 m in diameter and about 400 m long. Each penstock is provided with a surge tank which introduces additional hydraulic resistances.

The main object of the study was to determine if, as a result of the planned increase by 5 m of the headwater level, it would be possible to avoid having to raise the surge tanks. The point is important, since an increase in the surge-tank height, even if only by 1-2 m, without interrupting the operation of the plant, would present considerable engineering difficulties.

The paper deals with three problems: water hammer in the turbine intake and in the diversion conduits, water-level fluctuations in the surge tank, and temporary nonuniform operation of the hydraulic units.

The water hammer in the turbine-feeding conduits and its propagation to the diversion conduit has been determined graphically for different hydraulic resistances at the junction between the penstock and the surge tank and for different closing times of the turbine guide vanes. In these calculations allowance has been made for the inertial pressure of the stream flowing into the surge tank.

In determining the maximum increase of the water level in the surge tank, allowance was made for the water hammer during the closing of the

turbine and graphoanalytical calculation methods were used. The problem of the influence of turbine-closing time on the maximum level increase in the surge tank has also been investigated.

The temporary nonuniformity in turbine running was determined both by approximate graphical methods, based on the actual operating characteristics of the turbine, and by more accurate methods based on the numerical integration of the differential equation for the rotation of the turbine mass when the water head varies with time.

More accurate calculation methods have been suggested for a series of problems involving nonsteady flow conditions.

The results of these investigations permit the following conclusions:

1. Calculation of the surge-tank characteristics, with allowance for variations in the velocity of the streamflow in the diversion conduit during the closing of the turbine, showed that the maximum water level in the tank does not reach its upper edge and the storage capacity of the tank does not drop below its critical value. It follows that it is not necessary to increase the height of the tank.

2. The magnitude of the shock pressure in the diversion conduits and in the penstock was determined for different headwater levels, coefficients of hydraulic resistance at the junction between the penstocks and the surge tank, as well as for different values of the closure speed of the turbine.

3. The temporary nonuniformity of turbine running was established for the most adverse operational conditions, which permits a more correct tuning of the relay for closing the emergency turbine gates.

INVESTIGATION OF OPERATIONAL CONDITIONS OF THE KAKHOVKA HEP TURBINE UNITS

Scientific Guidance: G.I. Krivchenko, Candidate of Technical Sciences, Lecturer

Research Team: E.V. Kvyatkovskaya, Candidate of Technical Sciences
V.M. Klabukov, Engineer
N.N. Arshenevskii, Engineer

The aim of this study was to clarify the causes of the interruptions in the continuity of flow through the turbine unit during a drop of the load, to develop methods for the calculation of noncritical regulating conditions, and finally, to check the guide vane–blade angle relationships and the allowable conditions for maximum power output of the turbine.

The study was divided into the following stages:

1) field tests at the Kakhovka HEP turbine units under conditions of a drop in the load;

2) field tests under conditions of uniform load;

3) model tests on a special experimental unit under conditions of steady operation;

4) investigation of the flow pattern in the turbine-wheel area;

5) theoretical calculation of the regulating schedules suited to the conditions at the Kakhovka HEP.

These investigations afforded a clearer insight into the processes developing within the water passages during the closing of the turbine.

It was found, both by field and by model tests, that during the closing of the guide vanes the turbine unit passes to the so-called pumping regime at which the pressure above the turbine wheel becomes smaller than that within the neck of the draft tube, causing a negative (upward-directed) axial force. Under certain conditions, such as short durations of guide-vane closure, the flow continuity between the guide vanes and the runner may be interrupted which leads to the appearance of a reverse water hammer, causing an increase in the load on the runner blades, which may fail by breaking.

Special investigations regarding the pressure distribution in the water passages of the turbine established a marked nonuniformity of pressure distribution in the area in front of the runner, a fact which points to discontinuities of flow near the runner hub, where minimum pressures are observed under different operational conditions.

The results of these investigations made it possible to establish for the first time the relationships at different speeds of rotation for the nonuniformity of pressure distribution for any relationship between guide-vane and runner-blade openings.

The investigation of the stream flow below the runner blades showed that the pressure in this area depends mainly on the discharge and is almost uniform over the whole cross section.

Methods have been developed for calculating regulation schedules for adjustable-blade turbines, making use of the characteristics obtained for normal, runaway, and pumping regime of turbine operation.

On the basis of field and experimental tests, as well as of theoretical calculations, the study gives recommendations for establishing noncritical regulation schedules for the case of the Kakhovka HEP.

Much attention is devoted to the problem of determining the shock pressure on the runner blades for the cases of streamflow discontinuity. Model tests established pressure and load distributions on the runner blades for many different values of water-flow velocities in the draft tube, as well as for different blade angles and peripheral runner velocities.

The results of this stage of the investigation permit the calculation of the shock pressure and the load for cases of flow discontinuity; these results may be used in designing the strength of the runner blades and of their bolt connections.

These calculations showed that the loads on the runner blades during reverse water hammer markedly exceed the values usually assumed, a fact which points to a real danger of blade failure in the case of flow interruptions and reverse water hammer.

Field investigation of the Karkovka HEP turbines were carried out under load to study the guide vane–blade angle relationship and the permissible operational conditions for maximum power output. The blade-control valve linkages obtained from tests of turbines operating under a propeller regime were found to differ markedly from model-test data: the curves of the field tests are shifted in the direction of greater values of the opening of the turbine guide vanes.

Investigations on the operational conditions for maximum power output showed that for H_0 = 16.5 m, the guaranteed capacity of the turbine ($N_{\text{guar.}}$ = = 58.500 kw) is exceeded in practice by about 1000 kw. In order to establish the permissible maximum capacity for a wide range of heads, a universal

graph has been plotted permitting the maximum opening of the guide vanes and the blade angle to be determined for conditions of cavitation-free operation of the turbine unit at different tailwater levels.

The field investigations were conducted jointly by the Leningrad Branch of the Gidroproekt, the VNIIG imeni B. E. Vedeneev and the ORGRES.

MISI IMENI V. V. KUIBYSHEV CHAIR OF WATER MANAGEMENT AND MARITIME PORTS

Head: Professor N. N. Dzhunkovskii, Doctor of Technical Sciences

WAVE ACTION ON LOW-HEAD FISH-POND EARTH DIKES

Responsible for Research: Professor N. N. Dzhunkovskii, Doctor of Technical Sciences

Research Team: B. A. Kulygin, Candidate of Technical Sciences, Junior Lecturer (Head of Team)
E. V. Kurlovich, Candidate of Technical Sciences, Junior Lecturer
A. F. Semenov
A. N. Maksimov
A. A. Zhegulev
Ya. I. Knok
A. V. Vilkov

The problem of the erosion of the upstream slope of low dams due to wave action can be handled either by allowing a certain amount of erosion and carrying out periodic repairs, or by providing a suitable slope protection. Such protection should answer two requirements: strength and low cost. The best solution in each case will depend on the intensity and the total extent of the erosion.

Since most low-head dikes are built of cohesive soils whose erosion processes are not yet sufficiently understood, the investigation concentrated on this type of soils.

Soil erosion due to wave action depends on a number of factors: initial inclination of the slope, compactness of the soil layers, and physicomechanical properties of the erosion *.

The action of wind-driven waves on unprotected soil slopes having an inclination of 1:2 to 1:6, is due to the impact of the broken waves and their swash and backwash.

Three stages may be distinguished in the erosion process: in the first stage (on smooth slopes) the crest of the broken wave attacks the soil slope forming a cavity below the water line (Figure 136). The swash and the backwash of the wave causes slower erosion of the area above the water line. In the second stage, with the widening of the erosion cavity (Figure 137), the wave shocks concentrate on its upper rim extending the cavity toward the bank, but the intensity of erosion slows down due to the damping effect of the water cushion filling the cavity, and the difference between the effects of the wave shocks and the backwash is gradually effaced.

* Water-level fluctuations which also affect the morphological changes of the slopes did not have to be considered in this case, in view of the necessity to maintain the water table in fish ponds at a constant level.

The third stage is characterized by a considerable extension, in depth, of the erosion cavity (Figure 138) in which the direction of the oscillatory motion of the water changes as in a closed vessel; the impact action of the waves lessens considerably and the erosion process decreases gradually until the water velocity reaches nonerosion values and the slope is stabilized, and resists further erosion. The resulting shope profile has the following characteristic features:

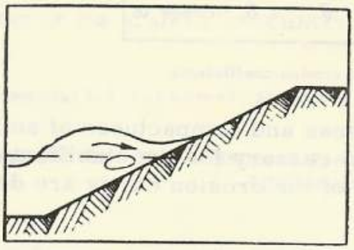


FIGURE 136. First stage of erosion processes

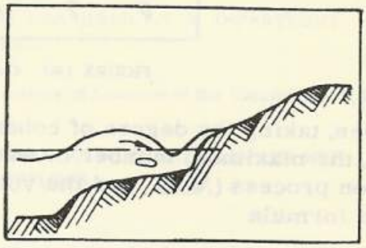


FIGURE 137. Second stage of erosion processes

a) small volume of deposited material compared with the volume of the eroded material;

b) the point of transition from the deposition to the erosion area occurs as a rule at a depth equal to the height of the effective wave;

c) the lower part of the erosion cavity has a very gentle slope; the upper part, on the contrary, ends in a steep, almost vertical face (Figure 139).

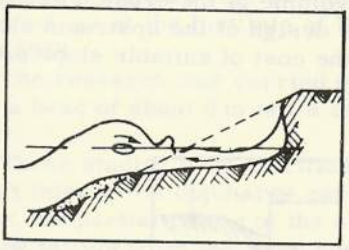


FIGURE 138. Third stage of erosion processes

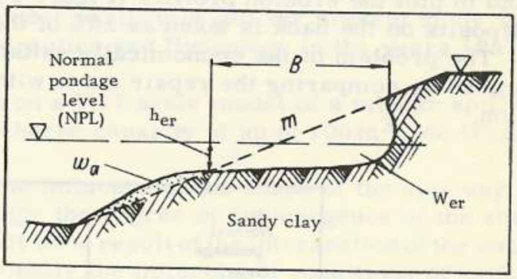


FIGURE 139. Profile of morphological slope changes after stabilization of erosion processes

In view of the lack of reliable soil-model methods, particularly in the case of cohesive soils, no models were used in the investigation. This was all the more possible as the maximum of wave-formation parameters in the wave-modeling flume are close to those of the actual waves. Extrapolation of the relations obtained ($K_{er}=f(m)$ and others is possible within small limits.

The intensity and extent of the wave erosion of the soil may be found by a method suggested in this work and based on empirical data.

To this end the wave parameters are determined from data on the wind velocity and duration. From the graph (Figure 140) plotted from data from tests on soils of varying cohesiveness, the erosion coefficient $K_{er}=f(m)$ is determined as a function of the initial slope inclination.

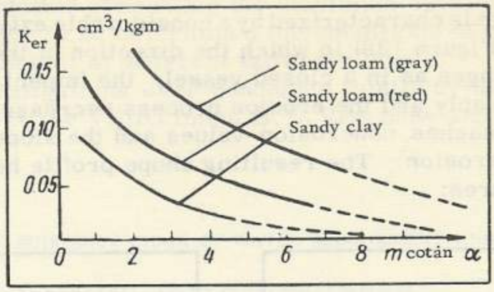


FIGURE 140. Curves of erosion coefficients

Then, taking the degree of cohesiveness and compactness of soil in the slope, the maximum number of waves necessary for the stabilization of the erosion process (N_{max}), and the volume of the erosion cavity are determined by the formula

$$W_{er} = 1{,}15\, K_{er} \cdot N_{max}\, h_{er}^{\,3} \cdot 10^{-5}\ \text{cm/run. cm},$$

where h_{er} = height of wave.

From the volume of the erosion cavity and the value of the slope inclination before and after erosion (m and m_1), respectively, it is possible to calculate the width of the erosion cavity

$$B = \sqrt{\frac{2W_p\, m_1 \cdot m}{m_1 - m}}$$

and to plot the erosion profiles (Figure 141) for which the amount of erosion deposits on the bank is taken as 50% of the volume of the erosion cavity.

The problem of the economically suitable design of the upstream slope is solved by comparing the repair costs with the cost of suitable slope protection.

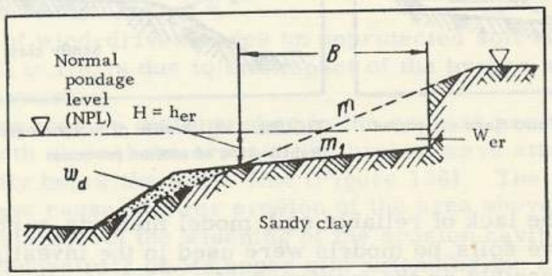


FIGURE 141. Schematic representation of erosion profiles

The study recommends to use, as an effective slope protection, a continuous soil-cement lining prepared by mixing cement grades "400" with the dike soil. Such a mix may be prepared on the spot and costs only 30% of the cost of a conventional slope lining. This type of slope protection has been subjected to the following tests: resistance to wave erosion, stability

of slope angle, resistance to crushing, resistance to crack formation at temperature changes from 15 to 50°C, and mechanical strength in water (resistance to wetting). The tests yielded satisfactory results.

The conclusions and recommendations are being used by Gidrorybproekt for the design of fish-pond structures.

IGiG OF THE ACADEMY OF SCIENCES OF THE UKRAINIAN S. S. R. DEPARTMENT OF HYDRAULICS

Head: Academician G. I. Sukhomel, Member of the Academy of Sciences of the Ukrainian S. S. R.

INVESTIGATION OF THE HYDRAULICS OF AUTOMATIC TUBULAR SPILLWAYS WITH RING-SHAPED OVERFLOWS

Responsible for Research: Academician G. I. Sukhomel, Member of the Academy of Sciences of the Ukr. S. S. R.

Research Team: I. L. Rozovskii, Senior Research Worker
P. K. Tsvetkov, Senior Research Worker

Tubular shaft spillways on relatively small ponds and water courses are gaining increasing acceptance due to the ease of their operation and also to the possibility of constructing them almost entirely by prefabrication methods. The Ukrgiprovodkhoz and other design institutes are developing standard designs for this type of water outlet, but they have been hampered by the lack of known and sufficiently established methods for the hydraulic calculation of such structures. In order to fill this gap, the present study was carried out at the Institute of Hydraulics and Hydrology in the years 1957 and 1958.

The research was carried out on a 1:17 scale model of a tubular spillway for a head of about 8 m and a discharge capacity of up to 100 m^3/sec (Figure 142).

These studies have clarified the influence of the shape of the spillway shaft inlet on its discharge capacity, the degree of submergence of the shaft inlet, the partial filling of the shaft as a result of the intersection of the water nappe falling from the inlet, and finally the influence of conditions of outflow, from the shaft into the tube, on the discharge capacity of the spillway.

The study also dealt with the widening of the water jet downstream from the spillway in the presence of such engineering features as a flaring overfall and outflow funnel with an opening angle of δ = 10 to 30°, a divergent stilling pool formed by a water-deflecting wall or by a bucket, a short flat diffuser placed at the conduit outlet, bucket-type energy dissipators, etc.

The study proposes methods for the hydraulic calculations of the sizes of shaft and conduit and of the structures for widening the nappe in the tail-water side. The nappe can be widened as much as ten times in the vertical direction, and as much as 15 times in the horizontal direction by providing: 1) a widening overfall followed by a funnel (angle of divergence δ = 10°); 2) a stilling pool with an opening angle of δ = 20 to 30° and a short diffuser conduit outlet.

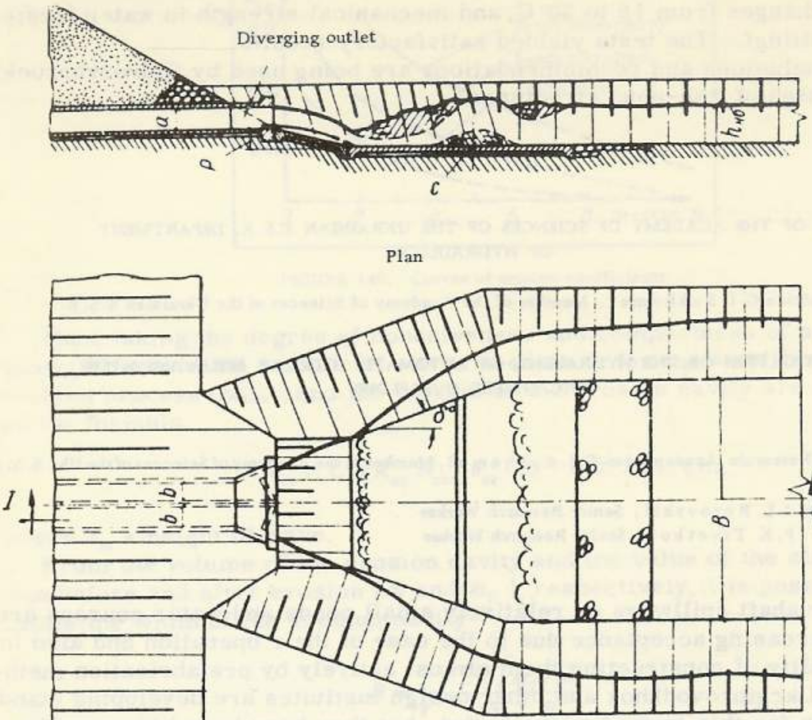


FIGURE 142. Model of tubular spillway

The conditions for widening the spillway water may be improved by reducing the Froude number at the outlet from the conduit. On the other hand, when passing from the bottom hydraulic jump to the surface jump formed behind the overfall, the conditions of the stream widening deteriorate sharply: the flow-velocity distribution along the tailwater channel becomes nonuniform and the flow,shooting.

A satisfactory widening of the stream in the horizontal plane by a factor of about 7, may be obtained by using a bucket-type energy dissipator consisting of an overflow wall of polygonal shape in plan, which closes the water space immediately downstream from the spillway conduit.

The methods presented in this study for calculating tubular spillway shafts permit a correct and more economical design of the structure, which prevents scouring of its downstream portion as well as of the tailwater channel. All the structures including those in the tailwater channel may be made of precast elements which permit a high degree of mechanization of construction work. The automatic operation of these structures ensures their maximum reliability and easy maintenance

The results of these investigations will be used by the Ukrgiprovodkhoz and other institutes as a basis for the design of standard tubular shaft spillways.

VERTICAL VELOCITY DISTRIBUTION IN A FLAT UNIFORM STREAM, AND HYDRAULIC RESISTANCES IN CONCRETE-LINED CANALS

Responsible for Research: Academician G. I. Sukhomel, Member of the Academy of Sciences of the Ukr. S. S. R.

Research by: I. A. Rodionov, Junior Research Worker

The problem of distribution of longitudinal average flow velocity at various depths in a turbulent stream of uniform section is of great practical importance in solving problems connected with hydraulic resistances, load transportation, transversal currents, etc. It has also scientific significance for the understanding of the phenomenon causing hydraulic resistances. The problem which is the subject of this research is therefore of great importance for the study of the hydraulics of open channels.

The study has been divided into several stages:

a) formulation of the problem of flow-velocity distribution and hydraulic resistances in open channels and critical analysis of existing methods for determining hydraulic resistances;

b) description of experimental techniques and of test results, and evaluation of the degree of accuracy attained;

c) analysis of obtained results, and recommendation of methods for determining hydraulic resistances in concrete-lined conduits.

The following conclusions have been drawn from these investigations:

1) the closest agreement with experimental data on flow-velocity distribution is given by the logarithmic and exponential curve and the worst agreement by Bazin's parabola and Karaushev's ellipse;

2) there are no grounds for considering the so-called Karman universal constant to be really a constant;

3) the extension of the relationships for pipes to open channels as is done by certain researchers is not justified and leads to erroneous results;

4) hydraulic resistances in concrete-lined canals are not governed by the Nikuradze-Zegzhda laws; therefore Zegzhda's hypothesis of the granular nature of such linings is not justified and the existing methods for calculating the hydraulic resistances of concrete water conduits cannot be accepted as satisfactory;

5) each type of canal surface has its corresponding curve of "smooth resistance".

The adoption of the author's recommendations and conclusions, which are fully supported by experimental data and by theoretical considerations, substantially increases the accuracy of the hydraulic calculations of concrete water conduits.

IGiG OF THE ACADEMY OF SCIENCES OF THE UKRANIAN S. S. R.
DEPARTMENT OF HYDRO ENGINEERING

Head: Professor B. A. Pyshkin, Doctor of Technical Sciences, Associate Member of the Academy of Sciences of the Ukrainian S. S. R.

DREDGED CUTS. PROBLEMS OF DESIGN

Head of Research: B. A. Pyshkin, Associate Member of the Academy of Sciences of the Ukrainian S.S.R.

Responsible for Research: S. V. Rusakov, Candidate of Technical Sciences, Senior Research Worker

Research Team: B. A. Pyshkin, Associate Member of the Academy of the Ukrainian S.S.R.
S. V. Rusakov, Candidate of Technical Sciences, Senior Research Worker
V. L. Maksimchuk, Junior Research Worker
Yu. N. Sokol'nikov, Senior Engineer

The study presents the results of theoretical and field investigations whose aim was to improve and develop the methods for determining the alignment and the characteristics of dredged cuts, as well as of the earth structures built to prevent silting up of the cuts.

A large number of data referring to dredging works on the Dnieper rapids, carried out in the period between 1925 and 1950, were collected, sorted and processed.

Large-scale field investigations on the regime of rapids located in the region of periodically occurring backwater were conducted continuously for two years (except during the winter ice).

Three series of laboratory investigations were carried out. In two series, fixed-bed models were used to study the conditions of silt deposition in dredging cuts of different direction and cross section; the third series, carried out on a movable-bed model, dealt with the determination of a stable cross-sectional profile of the protective earth structures.

These investigations provided a method to design the alignment width, shape, and cross section of the dredged cuts as well as the earth structures intended to protect the cuts and to ensure a better navigation channel.

The study will be useful not only to the Navigation Administration of the Ministry for River Transportation but also to other institutions dealing with river-navigation problems. The study, which may also be useful as a reference book, has been approved for publication by the Scientific Council of the Institute of Hydraulics and Hydrology.

LABORATORY INVESTIGATION OF HYDRAULIC-FILL BREAKWATERS MADE OF LOCALLY AVAILABLE SOIL

Head of Research: B. A. Pyshkin, Associate Member of the Academy of Sciences of the Ukrainian S. S. R.

Responsible for Research: S. V. Rusakov, Candidate of Technical Sciences, Senior Research Worker

Research by: V. N. Sidorchuk, Junior Research Worker

The aim of this investigation was to work out recommendations for the design and construction of hydraulic-fill breakwaters made of locally available soil on the storage reservoirs of the Dnepropetrovsk basin.

The study analyzes the existing methods for calculating the protective revetments of the slopes of earth-fill hydro structures and for designing nonprotected slopes of earth-fill structures built of noncohesive soil and subjected to frontal wave attack. The advantages and shortcomings of these methods are examined and the main objective of the study is defined, i. e. the development of engineering methods for designing the riprap revetment of earth hydro structures, and to refine the existing methods for designing unprotected slopes of small inclination. A characteristic feature of the new method recommended in this study is the assumption that all types of protection considered (riprap of varying size, and nonprotected slopes) are in a state of so-called dynamic equilibrium, in which the protective elements of the upper layers, under the action of the waves, are liable to shift along the slope relative to some central position without distributing the over-all stability of the protective layer as a whole. Experiments were carried out in a wave flume 30 m long, 0.75 m wide and 1.05 m deep. The basic parameters of the waves used in these tests varied as follows: height h = 6.5 to 22.8 cm; wave length L = 104 to 271 cm; relative wave length $\frac{L}{h}$ = 8.2 to 28.6.

The investigations were carried out on material of seven different size grades: sand, d_{av} = 0.23 and 0.17 mm; crushed stone, d = 3.32, 2.58, 2.03, 0.96, and 0.24 cm (here, d_{av} = average diameter of grain, as determined by sieve analysis; d = size of an equivalent stone cube).

The interpretation of the model results was made out on the basis of gravitational similarity laws. The tests were conducted on erodible models by placing at the end of the flume a slope model having an initial inclination m_0 = cotan φ, where φ is the angle of internal friction of the material in the saturated state. Waves of given parameters were then discharged through the flume and a stable slope was determined as a function of the wave parameters and the size grading of the material.

On the basis of the data obtained, the dimension of the basic elements of the protective structures were determined. Formulas were thus obtained for the upper and lower limits of erosive wave action, the coefficient of beach slope, and the coefficient of submerged surface of the bank.

Using this calculation method, one can compare the effectiveness of various types and designs of slope protection. It has been used by the design and construction institutes of the Main Administration of the Dnieper River Navigation, belonging to the Council of Ministers of the Ukranian S. S. R

DREDGES FOR MAINTENANCE OF NAVIGATION CHANNELS IN RIVERS

Head of Research: Professor B. A. Pyshkin, Doctor of Technical Sciences, Associate Member of the Academy of Sciences of the Ukrainian S. S. R.

Research by: G. B. Dokukin, Engineer

The purpose of these investigations was to test a new design of hydraulic dredges intended to reduce the maintenance cost of navigation channels at sandy river shoals, and to avoid the need of deepening the channels by blasting, as this practice is very harmful to the fishing industry.

The design of the new dredge was developed by G. B. Dokukin and tested by him under laboratory conditions in 1957. The first experimental dredge was built in 1958, and preliminary field tests were made in October of the same year.

The working principle of the new dredge, as distinct from conventional types, is based on the use of the combined energy of the stream current and of a steam or diesel tug for scouring the sandy bottom and removing the suspended bed load.

The experimental dredge (Figure 143) consists of a towed barge to which are attached two guiding units consisting of steel frames (3) and flat steel sliding plates (4) 4 meters long. The plates slide in the grooves and can be raised flush with the bottom part of the frame, or lowered into the water to a depth h (Figure 143, b and c) by means of threaded rods and handwheels.

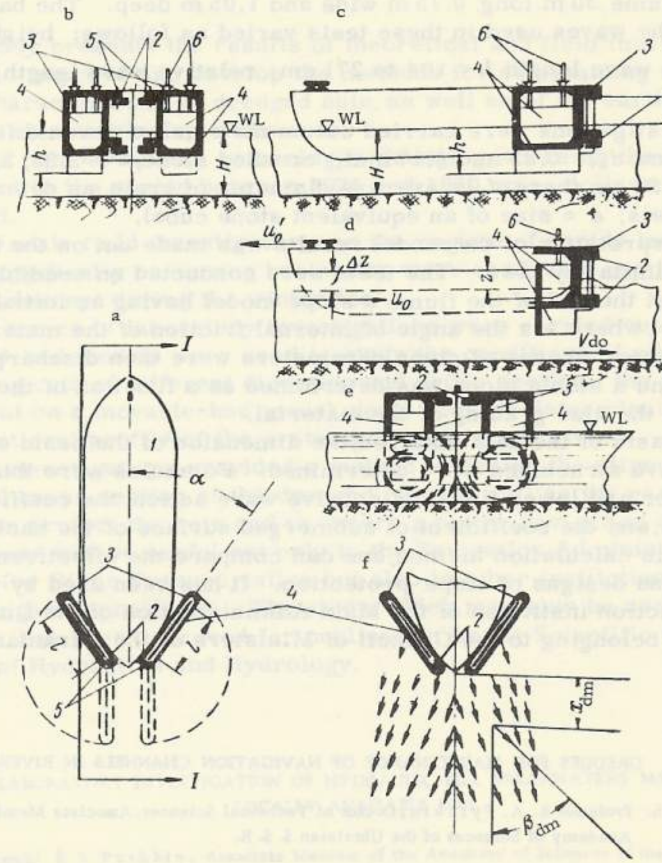


FIGURE 143. Experimental dredge

1–towed barge; 2–brackets; 3–steel frames; 4–flat steel plates; 5–fingers; 6–handwheel with threaded rod. WL– water level.

The steel frames are hinged to both sides of the barge stern by means of fingers (5) and brackets (2) welded to the barge body and may be rotated

about the vertical binges and be placed either in the position of the rudder or at an angle α to the center line of the barge (Figure 143, a).

In the working position the steel frames are fixed at the angle α, and the plates lowered to a depth h.

The deepening of sandy shoals is carried out by towing the dredge upstream and downstream over the shoal

While the dredge is towed over the shoal, the stream first flows around the guiding unit; then longitudinal-helical currents with a double transverse circulation are formed behind the stern (Figure 143, e, f) which increase the load-carrying capacity of the stream.

While the current flows around the guiding unit, the water flows out from underneath the gates, thus increasing the flow velocity near the river bottom v_{bv} (Figure 143, d) and giving rise to a scouring process. Immediately behind the guiding units appears a zone of reduced pressure and ascending currents which bring the river load into a state of suspension. Then, during the movement of the dredge, the river bottom is loosened and the suspended load is carried away by the longitudinal-helical currents. Under optimum operational conditions of the dredge, the resulting longitudinal currents should be able to completely carry away the loosened material.

Laboratory tests showed the optimum operational conditions of the dredge to occur for the following parameter values: $\alpha = 30°$ and $h = 0.8H$ [H = depth of water].

A towing power of 120 to 150 HP suffices for steam or diesel tugs in lowland rivers.

Field tests consisted in towing the dredge upstream and downstream over the shoals. The test results were recorded by a special team which measured the topography of the river bottom before and after dredging work.

It was found that a sand shoal surface 36 m wide and 200-250 m long can be lowered by 10-30 cm in 1 to 1.5 hours of net work, or, taking into account losses for turns, in about 2 to 4.3 hours. The actual output of the dredge amounts to 268 m^3/hour and the net cost of dredging, to 0.6 rubles/m^3 while, by using suction dredges, the net cost amounted to 0.93 rubles/m^3, and in blasting work, to as much as 2.7 rubles/m^3

The specific power consumption amounted to 0.67 HP/m^3/hr, while for suction dredges it reached 1.2 to 2.4 HP/m^3/hr.

The new dredge design has been adopted by the Main Administration and was put into operation in 1959. It may likewise be used in other rivers of the U.S.S.R.

INVESTIGATION OF THE STREAMFLOW JUNCTION BETWEEN THE HEADWATER AND TAILWATER SECTIONS OF BROAD-CREST SPILLWAY STRUCTURES WITH OR WITHOUT DENTATED APRON SILLS

Responsible for Research: N. N. Belyashevskii, Candidate of Technical Sciences, Senior Research Worker

Research by: N. G. Pivovar, Candidate of Technical Sciences, Junior Research Worker

The investigations were carried out during the years 1957 and 1958 within the framework of studies conducted at the Institute to develop methods

for the design of the tailwater section of different types of spillway structures.

The investigations covered three types of spillway structures:

1) broad-crest spillways with flashboard and plain apron;

2) as above but with a Rehbock-type dentated sill at the end of the apron;

3) as above but with bottom scour of d = 0.5, 1.0, 1.5 and $2.0H_3$ at the end of the deflecting floor.

The experiments were carried out in a 1.0 m wide flume in conformity with the conditions of the two-dimensional problem. In the models $H = H_3$ = 15 cm, and five different discharges could be obtained by removing one or more of the five stoplogs. The value of the Froude number at the contracted section decreased from 12.6 when one stoplog was removed, to 1.16 for the full opening of the outlet. The height of the sills was 0.15, 0.20 and 0.25 H_3.

The objective of the study was to arrive at formulas which, for given parameters – structure dimensions (H, H_3, a) and hydraulic flow characteristics (q, h_{n0}, h_n) – would make it possible to determine all the values required for determining the streamflow junction conditions of the actual bottom-erosive velocities, and the pressure drops characterizing the action of the streamflow on the protective structures.

The study also dealt with the problem of the energy-dissipating effectiveness of dentated sills for the low-head type of spillways considered.

The experiments, as well as the method of their interpretation, were based on the methods developed and published earlier by N. N. Belyashevskii.

Special tests on hard-bottom models made it possible to establish the relationships for changes in the length of the characteristic zones of streamflow junction and for the magnitudes of averaged river-bottom flow velocities and pressure drops. By means of these relations curves of average riverbed flow velocities and pressure drops could be plotted for the whole length of these characteristic zones.

The influence of increased turbulence of the bottom layers of the streamflow on the stability of the soil and of the stone protective linings, were studied on models in which the hard lining of the downstream apron and the funnel had been replaced by calibrated graded erodible material (from fine-grained sand to crushed stone), placed in special boxes. Coefficients of reduction of erosive flow velocity as a result of increased flow turbulence were established for different streamflow junction conditions and different hydraulic characteristics of the stream flow. The expression for the coefficient is $J_t = \frac{v_{m.e.}}{v_{n.t.e.}}$ where $v_{m.e.}$ is the mean bottom-erosive velocity in a flow of increased turbulence and $v_{n.t.e.}$ is the bottom-erosive velocity for the same material but for a flow of normal turbulence. The values of $v_{n.t.e.}$ for different types of noncohesive soils, including riprap, have been studied during the past years. Given the values of $v_{m.e.}$ and J_t for different stream cross sections at the streamflow junction zone it is easy to calculate $v_{n.t.e.} = \frac{v_{m.e.}}{J_t}$ and to calculate the stone size which will ensure the maximum stability. Assuming a certain factor of safety, the required size and weight of stones for the upper layer of the apron protection can be determined.

The study gives the graphs of $v_{m.e.}$, $v_{n.t.e.}$ and J_t for the downstream apron zone in spillways at which the head over the flashboards H = 4 m (case 1) and the rated head H_3 = 6 m (case 2).

In the first case no sill is necessary, while, for case 2, a dentated sill having a height $c = 0.2\ H_3$ is provided. The graphs were plotted for the following discharges:

1. Type 1 [see the beginning of this paper], $q_{max} = 13.5\ m^3/sec$.
2. Type 2, $q = 0.5 q_{max}$.
3. Type 3, $q = 0.25 q_{max}$.

In the absence of a bottom sill the stream joins the tailwater by a bottom-type submerged hydraulic jump (submergence ratio $\zeta = 1.11$ for q_{max}); in the presence of a bottom sill, the junction is by a surface-type submerged jump ($\zeta = 1.22$ for q_{max}). The graphs clearly show the velocity pattern in the lower stream layers close to the apron zone for both these cases, and they illustrate the following conclusions drawn from these investigations.

1. With plain aprons, streamflow junction occurs near the bottom of the stream. In this zone the average flow velocity $v_{m.e.}$ at the upstream end of the apron will be 1.5 to 4.5 times higher than at the downstream end. This lessening of the velocity increases with the degree of stream turbulence (i. e. of the Froude number) in the contracted stream section, and also with the increase in the degree of submersion of the hydraulic jump. It is even more marked due to the fact that at the upstream end of the apron $v_{n.t.e.}$ may be as much as twice $v_{m.e.}$ (for Froude number $Fr = 10$ to 12). With the decrease of the backwater pressure and the Froude number (when the dam-bay opening is increased) the turbulence of the stream diminishes. The coefficient J_t increases while the influence of more intensive turbulence becomes smaller. With the transition to the undulated jump ($Fr < 3.0$) when rollers and separation zones in the stream disappear, J_t approximates unity over the whole length of the downstream apron.

It can be seen that the erosive capacity of the stream is enhanced both by the increase in the average bottom flow velocities and the increased turbulence. For the types of spillways considered here, it is necessary to adopt as a calculation basis for the protection of the upstream and of the apron, not the maximum water discharges, but much smaller, intermediate discharges. On the other hand, for the downstream end of the apron, and the scouring funnel immediately below it, we should adopt the maximum water discharges as a calculation basis.

2. The main function of a dentated sill is to transfer the bottom flow conditions near the stilling pool to surface flow conditions at the apron further downstream. Downstream from the sill the surface flow regime is similar to that of spillway faces with upturned buckets. For the type of spillways considered, the surface flow conditions downstream from the sill (in view of the small Froude numbers) have a mixed surface-bottom regime. In the zone of the undulated jump ($Fr < 3.0$) the sill has only a local effect on the streamflow and no roller is noticed. In the zone of the surface submerged jump the average bottom velocities below the sill over the whole length of the flow-junction area are lower than the normal velocities at the end of this zone. The zone of velocity equalization decreases to about half its previous length. The sudden decrease in the average bottom velocities is accompanied by a sharp increase in the turbulence of the lower layers. Nevertheless the bottom velocities at the upper end of the apron are markedly lower than the comparable values without a bottom sill. The lower velocity and the smaller scouring action of the stream permit the use of a lighter protective lining in this area.

3. If there is a scouring funnel with a depth of $a > 0.5H_3$ downstream from the dentated sill, the effect of the sill on the hydraulic pattern of the stream becomes relatively small. The surface flow pattern appears at the main sill while the hydraulic characteristics differ but little from those existing downstream from conventional spilling floors. For funnels with $a > 0.5 H_3$ the addition of a sill reduces scouring by 5 to 20%.

The results of these investigations will be useful in the design of the tailwater area of low-head spillways and run-of-river dams. At present they are being used by several design institutes in Kiev. The study is being prepared for publication.

IGiG OF THE ACADEMY OF SCIENCES OF THE UKR.S.S.R.
DEPARTMENT FOR MECHANIZATION OF HYDRO-CONSTRUCTION WORK

Head: N. A. Silin, Candidate of Technical Sciences

INVESTIGATION OF HYDRAULIC-PRESSURE TRANSPORT OF SOIL IN UNSILTED OR PARTLY SILTED HORIZONTAL AND VERTICAL PIPELINES

Responsible for Research: N. A. Silin, Candidate of Technical Sciences

Research Team: N. A. Silin, Candidate of Technical Sciences
S. G. Kobernik, Engineer,
I. A. Asaulenko, Engineer,
K. V. Diminskii, Engineer

The investigation of hydraulic transport of soil was carried out in order to obtain data for the operating conditions of large suction dredges. Among these data are losses of head during the flow of water and of water-soil slurry in large-diameter pipelines, and the characteristics of large suction dredges As not much is known regarding similarity criteria for the movement of slurries through pipelines it was not possible to use empirical data obtained in small-diameter pipes. In the years 1956 to 1958 field tests were made on the suction dredges types 1000-80, 500-60, 300-40, and 12NZU, operating at the construction site of the Kremenchug HEP. The aims of these tests were as follows:

A) the study of hydraulic resistances for the flow of water or slurries with a density γ_s up to 1.30 ton/m^3 and a mean sand-grain diameter $d_m =$ = 0.30 mm:

1) in river-side pipelines, 410, 614, 800, and 900 mm in diameter for flow velocities from 2.5 to 8.5 m/sec;

2) in floating pipelines, 510, 614, 700, and 800 mm in diameter for different turn angles of the ball joints;

3) in bell-and-spigot pipes, 614 mm in diameter, laid on the hydraulic-filling site (not on trestles);

4) in the fittings of pressure pipelines, such as branch pipes, venturi tubes, ball joints, etc.;

B) internal-wall roughness in steel pipelines, both new and after prolonged use;

C) characteristics of suction dredges of the 1000-80, 500-60, 300-40, and 12NZU types when pumping water or slurry;

D) operation of control and measurement instruments: venturi tubes for flow-rate measurement, consistency meters for determining the density of slurry flowing through a horizontal section of the pipeline, by measuring the head loss between the beginning and the end of the pipe, as well as of slurry flowing through an inclined section of the pipeline, by measuring the head loss over a certain length of pipe.

The hydraulic resistances J in shore lines were determined as a function of the average velocity (v), the slurry density (γ_s) and the pipe diameter (D). Thus, $J = f(v, \gamma_s, D)$. For this function the basic parameters of hydraulic transport, J, v, γ_s had to be determined for each pipe diameter.

For plotting the graphs of operating characteristics of the suction dredges during the tests, the following parameters were determined: delivery Q, manometric (discharge) head H_m, suction lift H_s, power required N, density of slurry γ, and the rotation speed of the unit. At constant rotation speed the total head H, and the power consumption were determined as a function of Q and γ_s.

The parameters of the hydraulic-transport system were measured by self-recording instruments, which is of particular value for suction dredges operating under varying conditions.

The following conclusions were drawn from the test results:

1. During the flow of water through shore pressure pipelines, the hydraulic resistances are on the average smaller by 50% than the losses in new steel pipes as calculated by F. D. Shevelev for rectilinear pipelines.

2. The formula $J_s = J_w \cdot \gamma$, recommended by many authors, is valid for pipelines of a given diameter and density of slurry only for a definite flow velocity v_1. For $v > v_1$ or $v < v_1$, this formula is no longer valid.

3. Head losses in floating pipelines are 1.8-2.7 times greater than in river-side pipelines, and depend on the turn angle in the ball joints.

4. Head losses in bell-and-spigot pipelines of 614 mm diameter exceed the losses in shore pipelines by as much as 1.5 times.

5. The local resistance coefficients in branch pipes are 1.5 times larger for slurry than for water.

6. The roughness (i. e. mean height of irregularities) of internal pipe surfaces through which slurry has been conveyed for a long time is on the average 0.013 mm, which is almost a tenth of the roughness of new steel pipes.

7. The actual deliveries of suction dredges (for the rated heads) are shown in the table below. It can be seen that the actual delivery is larger than the design values obtained on the basis of tests with water.

TABLE

No.	Type of suction dredge	Rated head, m	Delivery, m^3/hr	
			Design	Actual
1	1000-80	80	10,000	14,000
2	500-60	60	5,000	8,750
3	300-40	40	3,000	4,000

8. The characteristics of the 500-60 and 300-40 type suction dredges feeding slurry pipelines differ from the characteristics for water pumping, as can be seen from Figure 114: the curves $H=f(Q, \gamma_s)$ for slurry have a steeper slope than $H=f(Q)$ for water. As a result, H_h (delivery head) developed by the suction dredges of the above types working on slurry becomes in certain cases (for relatively small deliveries) larger than H_w (for water); in other cases (for larger deliveries) it becomes smaller ($H_s < H_w$).

9. The increase in the slurry density for a small delivery of the dredge increases the suction lift, whereas for large deliveries the suction lift changes only slightly, but the delivery drops sharply.

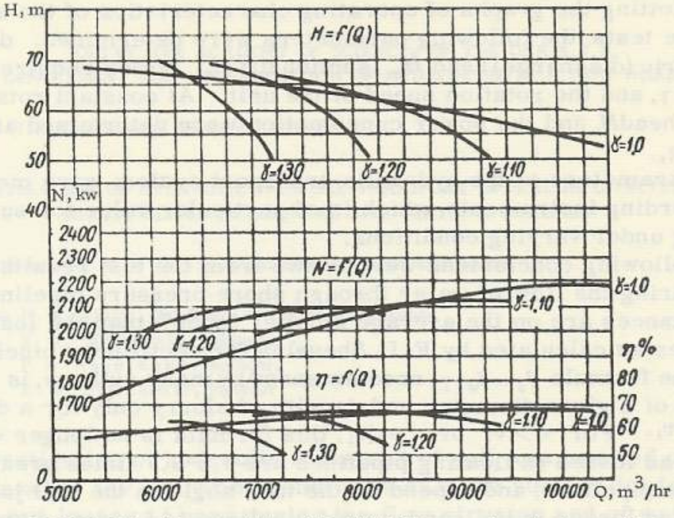


FIGURE 144. Characteristics of the 500-60 type suction dredge working on water and on slurry

The results of these investigations were submitted to the institutions interested (All-Union Trust "Gidromekhanizatsiya", etc.) and may be used for the calculation and selection of suitable operating conditions for suction dredges.

INVESTIGATION OF HYDRAULIC-PRESSURE TRANSPORT OF SMALL-SIZE COAL THROUGH PIPELINES

Head of Research: N. A. Silin, Candidate of Technical Sciences

Research Team: G. K. Vitashkin, V. N. Kondakov and I. I. Stavbun, Engineers

The construction of the first Soviet experimental pilot coal-transport pipeline 61 km long, from the Novo-Volynsk mines to the Dobrotvorskii thermal power plant started in 1958.

The designers encountered a series of basic problems concerning the choice of the main parameters of the system. Questions also arose relating to its reliable operation.

The Institute of Hydrology and Hydraulic Engineering carried out in 1958 a series of experimental investigations on the hydraulic transport of coal for conditions similar to those obtaining at this pipeline.

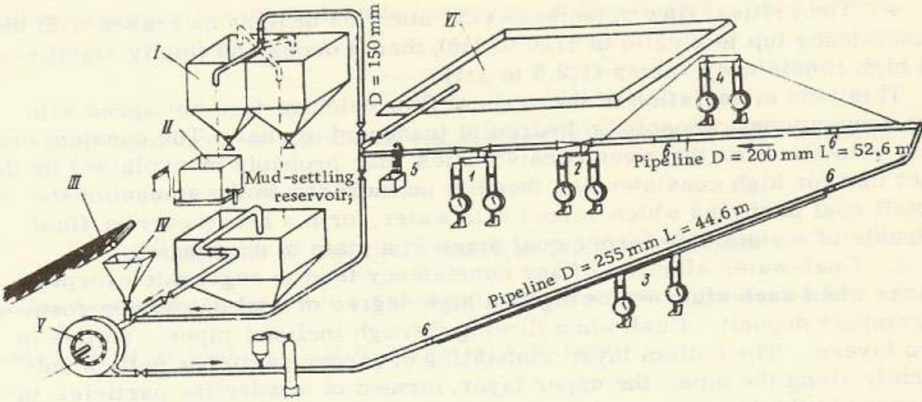


FIGURE 145. Layout of the test unit

I - measuring vessels, 13.2 m^3; II - weighing vessel of 1 m^3 capacity; III - ST-6 conveyor; IV - charging hopper; V - suction pump for slurry; VI - settling tank, 100 m^3

1-discharge measurement by a 200/150 venturi tube; 2-discharge measurement by a 200/130 venturi tube; 3-pressure gages for measuring head losses in a L = 29 m stretch of a D = 255 mm pipeline; 4-pressure gages for a L = 30 m stretch of a D = 200 mm pipeline; 5-water-coal slurry sampler for determining the size grading of coal; 6-transparent inserts for determining the critical flow velocity and the layer thickness of the silt deposits.

During these investigations the following problems were studied:

a) hydraulic resistances during the transport of coal having size grades of 0.6 and 0.25 mm in pipelines of 202 and 255 mm diameter, for a slurry consistency of solid-to-water ratio (by volume) 1:10-1:1 and 1:2.5-1:1.5, respectively.

b) conditions for conveying coal of 1-6 mm size grades with a maximum possible consistency of the water-coal slurry;

c) critical flow velocities for the conditions mentioned in paragraphs a, b;

d) variation in the coal size grades during transport;

e) processes of slurry settling during the transport through the horizontal and inclined stretches of the pipeline at high slurry consistencies.

The investigations were carried out on a test unit of the institute, redesigned to suit the conditions of hydraulic coal conveying. The layout of the test unit can be seen in Figure 145.

The test results permit the following conclusions:

1. Long-distance conveying of coal slurries through pipelines is technically feasible and differs but little from the hydraulic transport of other solid materials.

2. The maximum permissible slurry consistency consistent with normal haulage conditions corresponds to a solid-to- water ratio of 1:1 by volume. A slurry of such consistency represents a dense paste which, nevertheless, may be easily conveyed through pipelines.

3. The paper gives tables and charts for the determination of hydraulic resistances during the transport of 0.6 and 0.25 mm coal size grades in pipelines of 202 and 225 mm diameter. One of such charts is shown in Figure 146.

4. The critical flow velocity of coal slurries at first increases with the consistency (up to a ratio of 1:10 to 1:5), then it drops and finally stabilizes at high consistency values (1:2.5 to 1:1).

This kind of variation of the critical flow velocity does not agree with the conventional concepts on hydraulic transport of coal The constant critical velocities for the given consistencies may probably be explained by the fact that for high consistencies there is an increase in the amount of the small coal particles which, mixed with water, form a heavy carrier fluid capable of maintaining larger coal sizes in a state of suspension.

5. Coal-water slurries of any consistency tend to segregate except in cases when such slurries, owing to a high degree of coal saturation, form a compact deposit. Coal, when flowing through inclined pipes, settles in two layers. The bottom layer, consisting of larger particles, settles uniformly along the pipe; the upper layer, formed of powderlike particles, increases in thickness toward the lower section of the pipe. Under certain conditions of consistency and slope, this may lead to the clogging of the elbows at the pipeline turns. At a slope angle of 15°, the elbow tends to clog even at a consistency of only 1:5; an elbow at an angle of 10° clogs at a consistency of 1:2.5; an elbow of 5° does not clog even at 1:1.3.

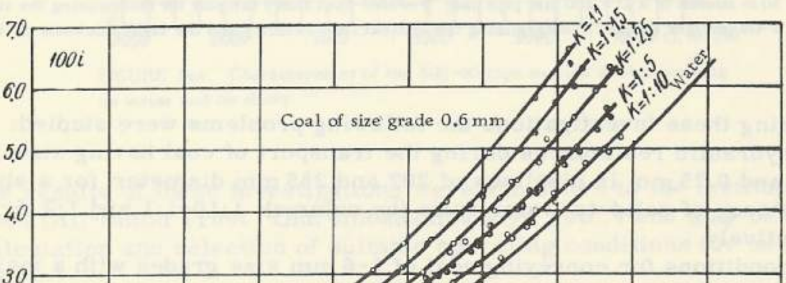


FIGURE 146. Relation of hydraulic gradient to the water-coal slurry velocity in a pipeline of 255 m diameter

6. Intense comminution of coal during transport is noticed during the first hours of operation of the unit; the process of comminution markedly slows down with time; the amount of small sizes (particularly under 0.25 mm)

considerably increases and the proportion of larger fractions decreases. Water-coal slurries of higher consistency are more exposed to intense comminution.

7. The study gives a method for determining, by means of charts, formulas, and tables, the optimum flow velocity and consistency required to ensure the best operating conditions for the hydraulic transport.

The investigation results were used in the design of the Novo-Volynsk semi-industrial coal-slurry pipeline and may also be useful in the design of other pipelines for the hydraulic conveying of small-size coal slurries.

HYDRAULIC LABORATORY OF THE IGiG OF THE ACADEMY OF SCIENCES OF THE UKR.S.S.R.

Head: M. M. Didkovskii, Candidate of Technical Sciences

INVESTIGATION OF SHOOTING STREAMS IN TAILWATER SECTIONS

Responsible for Research: Academician G. I. Sukhomel, Academy of Sciences, Ukrainian S. S. R.

Research Team: M. M. Didkovskii, N. G. Poznyaya, B. M. Egidis

The water flowing over the spillway into the tailwater section assumes the character of a shooting flow, in which the stream emerging from an overflow structure and reaching the tailwater widens very slowly, and turns toward one of the river banks. Between the spillway stream and the river banks appear several eddy zones in which the flow velocity approaches the velocity of the spilling stream. The velocity of the spilling stream and of the eddy zone is greater than the normal flow velocity of the river while the stream turbulence increases as a result of formation of interfaces within the water body during widening of the spilling stream. These phenomena cause considerable scouring in the tailwater section as has been observed in a number of small and large hydro plants. This is why the study of shooting streams in the tailwater is of particular importance.

The investigations covered two stages:

1) study of shooting-flow processes;

2) development of measures for their prevention.

The study was carried out on a laboratory model with unilateral widening of the stream flow [similar to actual conditions]. The distribution of actual flow velocities was measured by two-element strain-gage transducers. The measured values were recorded on an MPO-2 oscillograph.

In order to measure the actual flow velocity, the measuring scheme was supplemented by an electronic amplifying computer which ensured the linearization of the relationship between the flow velocity and the recording on the oscillographic tape.

The laboratory research was supplemented by field investigations on shooting streams in the tailwater of a small hydro plant in the Poltava region, where the flow velocity was measured by a current meter suspended by two wires, the measured values being continually recorded on an MPO-2 loop oscillograph.

The results of the laboratory studies stressed the necessity for further refining the calculation methods developed by P. M. Levi. Field investigations established the intensity range of large-scale turbulence in the area of spilling-stream widening; the amplitude of large-scale velocity fluctuations in this area was found to be much larger than that in the area downstream from the flow widening. These investigations also established the important role of spilling-stream velocity fluctuations in the process of channel scouring in the area of flow widening downstream from the spillway.

The results of these investigations may be instrumental in the development of suitable means for protection of tailwater structures but, at the same time, point to the necessity of carrying out further research into the processes of energy dissipation in the tailwater area.

InEG OF THE ACADEMY OF SCIENCES OF THE ARMENIAN S. S. R
DEPARTMENT FOR HYDRAULICS OF STREAMS AND STRUCTURES

Head of Research: Professor A. K. Ananyan, Doctor of Technical Sciences

CALCULATION OF THE LONGITUDINAL PROFILE OF A RIVER CHANNEL FOR CONTINUOUS VARIATIONS OF ITS BASE LEVEL

Responsible for Research: Professor A. K. Ananyan, Doctor of Technical Sciences

Research Team: Professor A. K. Ananyan, Doctor of Technical Sciences
M. S. Pokhsraryan, Junior Research Worker

The problem of calculating the longitudinal profile of a river channel for a continuous rise or fall of the level at its mouth (this variation being governed by an arbitrarily taken law) arose in connection with the lowering of the level of Lake Sevan which forms the base level for its tributaries. A similar pattern may also be noticed in a river on whose course storage reservoirs are built: the level of such rivers may vary with the water level in the storage reservoir.

At present, plans are under way for erecting on the Sevan tributaries a series of bridges and other hydro structures. In determining the elevations for their foundations as well as other design parameters, allowance has to be made for possible variations in the base level of these tributaries.

The aim of this study was to derive formulas for calculating the longitudinal profile of a river channel at any time given the law of base-level variation, the size grading of the river-bed soil, the river discharge which causes changes in the river channel, the plan of the river and the initial longitudinal profile.

The problem can be tackled by the simultaneous solving of the following equations:

1) channel deformation

$$\frac{\partial G}{\partial x} + \frac{\partial z}{\partial t} = 0, \tag{1}$$

2) nonuniform streamflow

$$\frac{\partial h}{\partial x} + \frac{\partial z}{\partial x} = -\alpha \frac{\partial}{\partial x}\left(\frac{u^2}{2g}\right) - \frac{u^2}{C^2 h}, \tag{2}$$

3) load-transporting capacity of the river

$$G = S_{av} \cdot q = q\varphi\left[\frac{h}{d}, A\left(d\right)\frac{i\sqrt{ghi}}{\sigma\varkappa w}\right], \tag{3}$$

where: G = solid load carried by the turbulent stream;
S_{av} = average turbidity of stream along the river section;
q = discharge per unit width of the river;
z = datum line (elevation) of river bed (Figure 147);
h = stream depth;
u = average flow velocity along the river section;
C = Chezy's coefficient;
σ, d, w = relative density, diameter, and fall velocity, respectively, of the solid load particles;
$i = \frac{\partial z}{\partial x}$ = slope of river at the given site;
$\varkappa$ = Karman constant;
$A(d)$ = parameter dependent on d [not defined in the Russian original].

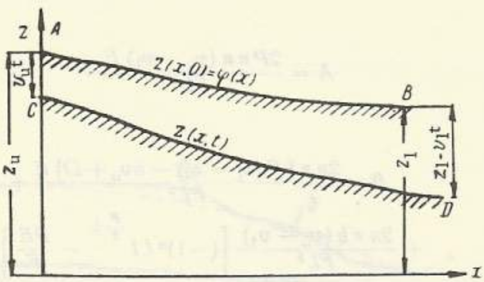


FIGURE 147. Graph showing deformation of river-channel bottom

Here it is assumed that the solid load passing at any moment at each site [gaging station] corresponds to the load-transporting capacity of the river at the given site and at the given moment.

The model and field tests carried out by the institute showed that the lateral (bank) erosion of the river channel is negligible compared with the channel-bottom erosion. This result permits us to solve the problem, considering it to be, as a first approximation, two-dimensional.

The load-transporting capacity of the stream is determined from M. Velikanov's formula

$$G = qS_{av} = q\,\frac{A(d)\,i\sqrt{ghi}}{\sigma\varkappa w} \tag{4}$$

By simultaneously solving equations (1)–(3) for $z(x,t)$ we obtain equation

$$\frac{\partial^2 z}{\partial x^2} + P\frac{\partial z}{dx} + b\frac{\partial z}{\partial t} + D = 0, \tag{5}$$

where

$$P = i\left[3h\left(1 - \frac{\alpha q^2}{gh^3}\right)\right]^{-1};\ D = P_i,$$

$$b = -2\,\sigma\varkappa\,w\left[3q\,A(d)\sqrt{ghi}\right]^{-1}.$$

The final solution can now be written as

$$z(x,t) = \frac{x}{L}(z_1 - v_1 t) + \frac{L-x}{L}(z_u - v_u t) +$$

$$+\left(\sum_{n=0}^{\infty} C_n(t)\sin\frac{n\pi x}{L}\right)\exp\left(\frac{P^2}{4b}t - \frac{P}{2}x\right), \tag{6}$$

$$C = \frac{A}{mb} - \frac{B}{mb} - \frac{z_\text{к}}{L}x - \frac{(L-x)}{L}z_u + \varphi(x);$$

$$A = \frac{2P\pi n(v_u - v_1)E}{FL^3}, \tag{7}$$

$$B = \frac{2n\pi\left[P(z_1 - z_u) - bv_u + D\right]E}{FL^3} +$$

$$+\frac{2n\pi b(v_u - v_1)}{FL^3}\left[(-1)^n L l^{\frac{P}{2}L} - \frac{PE}{F}\right], \tag{8}$$

$$m = \frac{n^2\pi^2}{bL^2} - \frac{P^2}{4b};\ E = \left[1 - (-1)^n l^{\frac{P}{2}L}\right];$$

$$F = \left(\frac{P}{2}\right)^2 + \left(\frac{n\pi}{L}\right)^2. \tag{9}$$

Formula (6) permits the longitudinal profile of the river to be computed for any time interval t.

Figures 148, 149 and 150 give a comparison of theoretical results with data obtained from model tests.

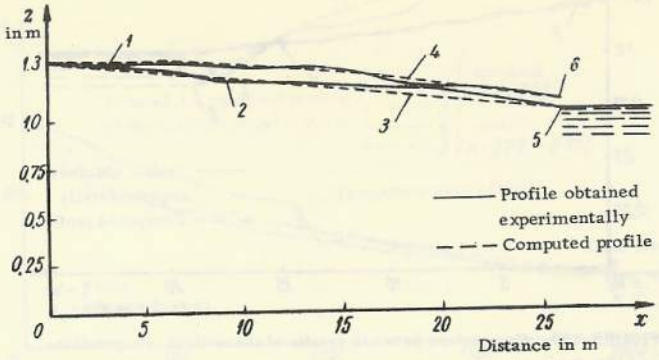


FIGURE 148. Comparison between results of theoretical computations and test data

1-site profile for $t = 0$; 2-river profile, determined experimentally, after the lowering of the base level for $t = 3$ hrs; 3-river profile determined by computation for $t = 3$ hrs; 4-approximate profile; 5-base level for $t = 3$ hrs; 6-base level for $t = 0$.

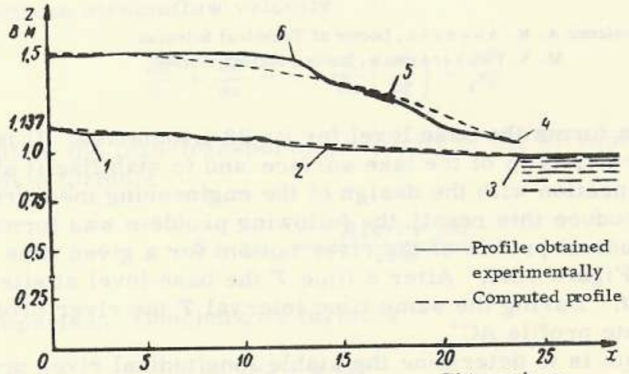


FIGURE 149. Comparison between results of theoretical computations and data obtained by experiments on channel models of the institute

1-river-bottom profile, determined after the lowering of base level, for $t = 452$ hrs; 2-river-bottom profile, found by theoretical computations, for $t = 452$ hrs; 3-base level for $t = 452$ hrs; 4-base level for $t = 0$; 5-approximate profile; 6-site profile for $t = 0$.

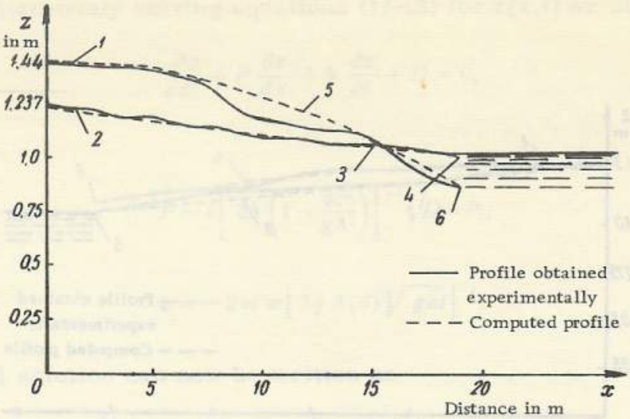


FIGURE 150. Comparison between results of theoretical computations and test data

1-site profile for $t = 0$; 2-river-bed profile, determined experimentally after the lowering of the base level, for $t = 324$ hrs; 3-river-bottom profile, determined by computation, for $t = 324$ hrs; 4-base level for $t = 324$ hrs; 5-approximate profile; 6-base level for $t = 0$.

ON THE STABLE LONGITUDINAL PROFILE OF A RIVER-CHANNEL BOTTOM

Responsible for Research: Professor A. K. Ananyan, Doctor of Technical Sciences

Research Team: Professor A. K. Ananyan, Doctor of Technical Sciences
M. S. Pokhsraryan, Junior Research Worker

Lake Sevan forms the base level for its 28 tributaries. It is intended to arrest the further drop of the lake surface and to stabilize it at a certain level. In connection with the design of the engineering measures that will be taken to produce this result, the following problem was formulated:

The longitudinal profile of the river bottom for a given time t is expressed by line AB* (Figure 151). After a time T the base level at site C* drops by the value H. During the same time interval T the river profile acquires an intermediate profile AC*.

The problem is to determine the stable longitudinal river profile between sites A and B if the elevation Z_u of the upper site and $(Z_1^0 - H)$ of the lower site remains constant. The following data must obviously be given: solid load carried by the stream from the upper stretches and passing through site A; size grade of the channel-bed soil; discharge of the river causing changes in the channel morphology; water level of the lake, or depth of water at site C. Here, it should be noted that the conditions of the problem do not require that the initial river profile AB (Figure 151) be known. This line has been mentioned only for a better understanding of the nature of the problem.

* [Sites A, B, C, and curves AB and AC are not designated in this figure; line AB is apparently the line of initial longitudinal profile shown in the figure.]

Since the lateral (bank) erosion of the river is negligible compared with the bed erosion, we may, as a first approximation, solve the simpler two-dimensional problem.

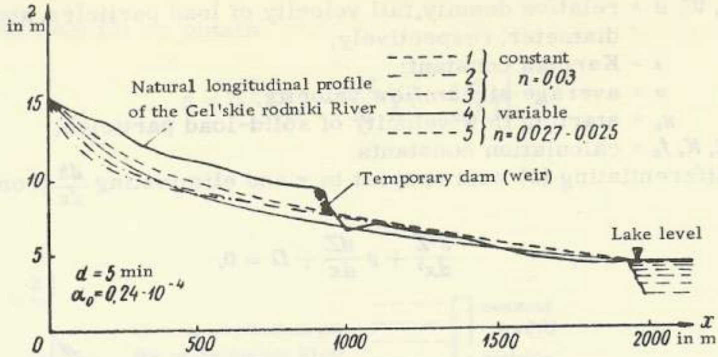


FIGURE 151. Curves of longitudinal river profiles

Determination of G by formulas of the following authors:
1 and 4-Velikanov; 2 and 5-Goncharov; 3-Egiazarov.

The problem is solved by applying jointly the following equations:
1) load-carrying capacity of river:

$$G = q\varphi\left[\frac{\cdot h}{d},\ A(d)\,\frac{i\sqrt{ghi}}{\sigma\varkappa W}\right], \tag{1}$$

2) nonuniform streamflow velocity

$$\frac{\partial h}{\partial x} + \frac{\partial Z}{\partial x} = -\alpha\frac{\partial}{\partial x}\left(\frac{u^2}{2g}\right) - \frac{u^2}{c^2 h}. \tag{2}$$

To calculate the load-transporting capacity of the stream we apply M. Velikanov's formula

$$G = qS_{\text{av}} = q\,\frac{R(d)\,i\sqrt{ghi}}{\sigma\varkappa W}, \tag{3}$$

and for comparison, Goncharov's formula

$$G = \frac{Cd}{u_0^{\,8}}\left(\frac{d}{h}\right)^{1/_{10}} u^3(u-u_0) \tag{4}$$

and Egiazarov's formula

$$G = qk\sqrt{i}\left(\frac{hi}{\sigma d f_0} - 1\right), \tag{5}$$

where G = solid load;

$i = -\frac{dZ}{dx}$ = river slope at gaging site;

h = stream depth;

σ, W, d = relative density, fall velocity of load particles, and particle diameter, respectively;

$\varkappa$ = Karman constant;

u = average streamflow velocity;

u_0 = starting flow velocity of solid-load particles;

$A(d), C, K, f_0$ = calculation constants.

By differentiating (3) with respect to x and eliminating $\frac{dh}{dx}$ from (2) we obtain

$$\frac{d^2Z}{dx^2} + p\frac{dZ}{dx} + D = 0, \tag{6}$$

where

$$p = i\left[3h\left(1 - \frac{\alpha q^2}{gh^3}\right)\right]^{-1}, \; D = p\frac{q^2}{C^2h^3}.$$

We linearize equation (6) by averaging p and D for small intervals of variation of h and i.

By integrating (6) for boundary conditions $Z(0) = Z_u$ and $Z(L) = Z_1$, we obtain

$$Z(x) = Z_u - \frac{Z_u - Z_1 - \lambda L}{1 - l^{-pL}}(1 - l^{-pL}) - \lambda x, \tag{7}$$

where

$$\lambda = \frac{q^2}{C^2h^3}. \tag{8}$$

Formula (7) has been derived assuming a constant roughness coefficient of the river channel.

If, however, we assume that between the upper and lower sites the roughness coefficient varies, e.g. by an exponential law $n = n_0e^{-\alpha_0x}$, we obtain

$$Z(x) = Z_u - \frac{M}{2\alpha_0p - 4\alpha_0^2}(1 - e^{-2\alpha_0x}) -$$

$$-\frac{Z_u - Z_1 - \frac{M}{2\alpha_0p - 4\alpha_0^2}(1 - e^{-2\alpha x})(1 - e^{-px})}{1 - e^{-pL}}, \tag{9}$$

where

$$M = \frac{q^2n_0^2p}{h^{10/3}}.$$

In a similar manner we obtained the relationships for $Z(x)$ by applying (4) and (5). For a constant roughness coefficient formula (9) remains unchanged except for the coefficient p.

When formula (4) is used this coefficient becomes

$$p=i\left[\ h\left(3-\frac{2u_0}{u}\right)\left(1-\frac{\alpha q^2}{gh^3}\right)\right]^{-1}. \tag{10}$$

With formula (5) we obtain

$$p=2i\left[\ h\left(3-\frac{\alpha d f_0}{h_l}\right)\left(1-\frac{\alpha q^2}{gh^3}\right)\right]^{-1}. \tag{11}$$

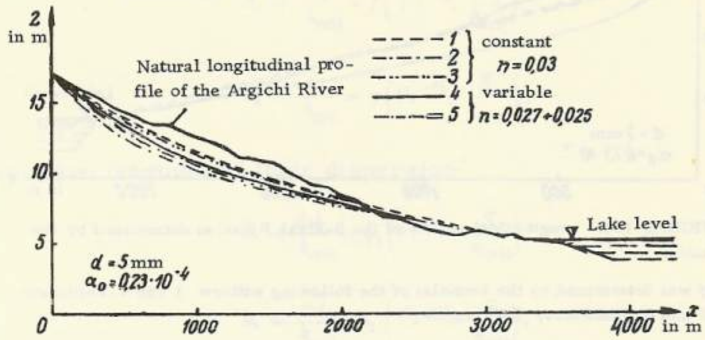


FIGURE 152. Longitudinal profile of the Argichi River determined by the author of this study

σ was determined by the formulas of the following authors: 1 and 4-Velikanov; 2 and 5-Goncharov; 3-Egiazarov.

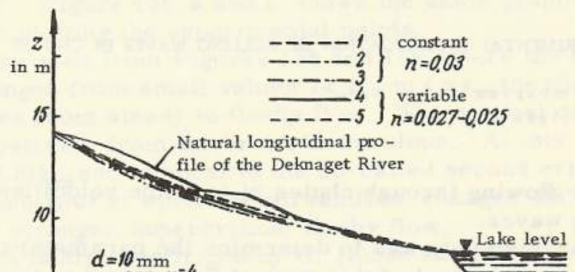


FIGURE 153. Longitudinal profile of the Deknaget River, as determined by the author

σ was determined by the formulas of the following authors: 1 and 4-Velikanov; 2 and 5-Goncharov; 3-Egiazarov.

Figure 152 illustrates the results of calculations by means of these formulas. The line in the same figure shows the actual natural longitudinal profile of the Argichi River, one of the larger tributaries of Lake Sevan. This profile has not yet been stabilized in view of the comparatively rapid drop in the level of the river mouth.

Similar calculations, carried out for other lake tributaries, are shown in Figures 152, 153 and 154.

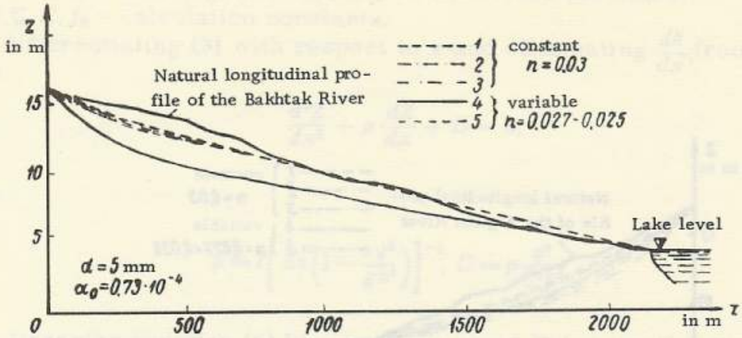


FIGURE 154. Longitudinal profile of the Bakhtak River, as determined by the author

a was determined by the formulas of the following authors: 1 and 4-Velikanov; 2 and 5-Goncharov; 3-Egiazarov.

If the present base level of the lake tributaries will not change, the pattern of the stable river profiles will probably assume a shape close to the curves shown in Figures 152 and 154.

We suggest to use in practice the formulas derived from expression (1) since they agree fairly closely with the results of field measurements of the river load.

EXPERIMENTAL INVESTIGATION OF ROLLING WAVES IN CHUTES

Research by: A. O. Gambaryan
N. N. Mailyan

Shooting water flowing through chutes at extreme velocities forms characteristic rolling waves.

The purpose of this study was to determine the parameters (depth, length, discharge, and wave velocity) of this type of flow, as a function of the stream and the channel parameters. This knowledge is indispensable to the hydraulic [and structural] design of chutes.

The following values were determined: the mathematical probability of the wave depth d_h and length d_λ, their dispersion, σ_h and σ_λ, as well as the distribution-probability curve for the wave parameters. The results of numerous experiments , and their verification according to Kolmogorov's

conformity criteria and coefficient χ^2, showed these probabilities to follow the normal Gaussian distribution law:

$$y = \frac{1}{\sigma\sqrt{2\pi}} \exp \frac{(x_i - d)^2}{2\sigma^2}, \tag{1}$$

where y = probability of the value under consideration;

x_i, $d = \frac{1}{n}\sum x_i$ = mathematical expectancy;

$\sigma^2 = \frac{1}{n}\sum (x_i - d)^2$ = dispersion of this value.

The following empirical relationships were obtained:

1) for wave depths and their dispersion

$$\bar{d}_h = \frac{d_h}{h_{\text{crit}}} = f\left(\mathrm{Fr}^{-1/3}, \frac{S_l}{h_{\text{crit}}}\right), \tag{2}$$

$$\bar{\sigma}_h = \frac{\sigma_h}{h_{\text{crit}}} = \varphi\left(\mathrm{Fr}^{-1/3}, \frac{S_l}{h_{\text{crit}}}\right); \tag{3}$$

2) for wave lengths and their dispersion

$$\bar{d}_\lambda = \frac{d_\lambda i}{h_{\text{crit}}} = \psi_1\left(\mathrm{Fr}^{-1/3}, \frac{S_l}{h_{\text{crit}}}\right) \tag{3}$$

$$\bar{\sigma}_\lambda = \frac{\sigma_\lambda i}{h_{\text{crit}}} = \psi_2\left(\mathrm{Fr}^{-1/3}, \frac{S_l}{h_{\text{crit}}}\right), \tag{5}$$

where S = distance from the upper end of the model;

Fr = Froude number;

$i = \sin\alpha$ = slope of channel bottom.

Figures 155, 156 and 157 show the curves for the dependence of the average wave parameters and their dispersion on the dimensionless model length, the hydraulic resistance of the channel, and the kinetic energy of the stream. In Figure 155, a and b all experimental data for $\bar{d}_h$ and $\bar{\sigma}_h$ are plotted on the separate graphs, as a function of the slope and the roughness of the channel. Figure 156, a and b shows the same graphs but superimposed and without plotting the experimental points.

As can be seen from Figures 155 and 156, where the slope of the channel bottom changed from small values $i < i_{\text{crit}}$ to $i > i_{\text{crit}}$ the character of the streamflow changes from steady to flashy flow. The critical depth occurs some distance upstream from the break in the slope. At this point the depth will be $(0.7$ to $0.8)h_{\text{crit}}$ and is equal to the so-called second critical depth (according to Vedernikov) at which the streamflow changes its flashy movement into a still stronger, supercritical flashy flow.

This fact is also confirmed by H. Rouse and other investigators.

Thus, at the initial part of the steep section of the chute the stream passes through the second, critical depth $h_{\text{crit}}^{\text{II}}$ assuming this is followed by a recession curve at the end of which the stream tends to acquire the depth h_0 corresponding to a uniform flow. It is approximately in this zone that the first visible waves appear. The process then passes into a marked mutual absorption of waves with their development both in height and length.

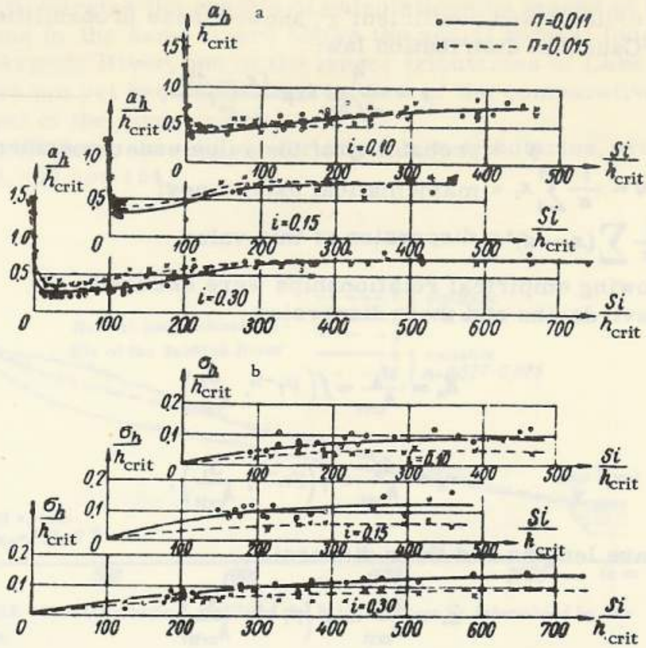
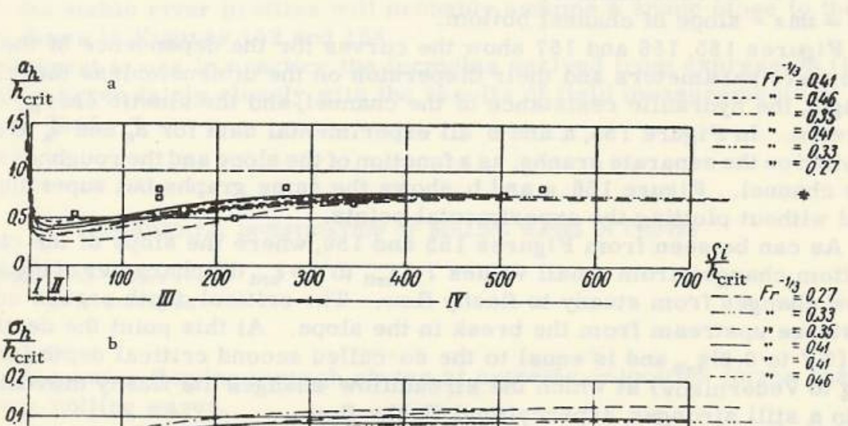


FIGURE 155. Relation of the average wave parameters and their dispersion to the dimensionless model length, the channel resistance and the kinetic energy of the stream

a-relation of $\bar{a}_h$ to the stream-channel characteristics for various values of the bottom slope; b-relation of dispersion σ_h to the same values.



* Observations made by Holmes ▫ Experimental points obtained for the spillway chute of the Gyumush HEP

FIGURE 156. Relation of the average wave parameters and their dispersion, to the dimensionless model length, the channel resistance, and the kinetic energy of the stream

a-curves showing the relation of $\bar{a}_h$ to channel and stream characteristics; b-curves showing the relation of $\bar{\sigma}_h$ to channel and stream characteristics.

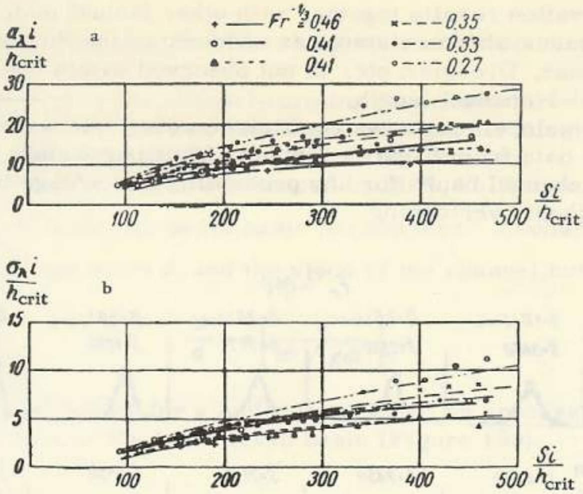


FIGURE 157. Relation of the average wave parameters and their dispersion to the dimensionless model length, the channel resistance, and the kinetic energy of the stream

a-curves showing the relation of $\overline{d_\lambda}$ to channel and stream characteristics; b-curves showing the relation $\overline{\sigma_\lambda}$ to channel and stream characteristics.

Further downstream, at the point where the ratio $\frac{S_i}{h_{crit}} \approx 300$, the mathematical expectancy of the wave depth $\frac{d_h}{h_{crit}}$ asymptotically tends toward a constant limit (0.7 to 0.8), i. e. to the second critical depth.

Here, it should be noted that, whereas the initial section of the curve $\frac{d_h}{h_{crit}} = f\left(\frac{S_i}{h_{crit}}\right)$ shows a clear distinction between the experimental points for slope and those for roughness, at large values of $\frac{S_i}{h_{crit}}$, this clear distinction disappears.

Figures 155, b and 156, b show the relation of the dispersion σ_h of the wave depths, to the stream and channel characteristics.

At the upper end of the channel model, where waves have not yet formed, ratio $\frac{\sigma_h}{h_{crit}}$ is nil.

With the increase in $\frac{S_i}{h_{crit}}$ the dispersion increases monotonically toward a definite constant value for each combination of channel slope and roughness.

The control points plotted in Figure 156, a were obtained from field measurements at the chute of the Gyumush HEP (for $i_{av} = 0.34$) and from Holmes' calculations (for $i = 0.70$).

Similar relationships have also been obtained for the mathematical expectancy of wave lenghts $\frac{d_\lambda i}{h_{crit}}$ and their dispersion $\frac{\sigma_\lambda i}{h_{crit}}$ (Figures 157, a and b).

As may be seen from Figures 156, b and 157, b, the dispersion of wave depths increases tending toward a certain definite limit for the corresponding value of the Froude number $Fr^{-1/3}$. This points to a certain difference between the wave parameters, this difference increasing further downstream.

The observation results together with other factual material show that the train of waves whose existence is assumed in the theoretical investigations by Thomas, Dressler, etc. is not observed within the rather wide limits of the model-channel length.

With known slope, roughness coefficient, water discharge, length of chute, and using the data from Figures 156 and 158, it is possible to calculate the height of the channel banks for any probability percentage of passage of water waves without overtopping.

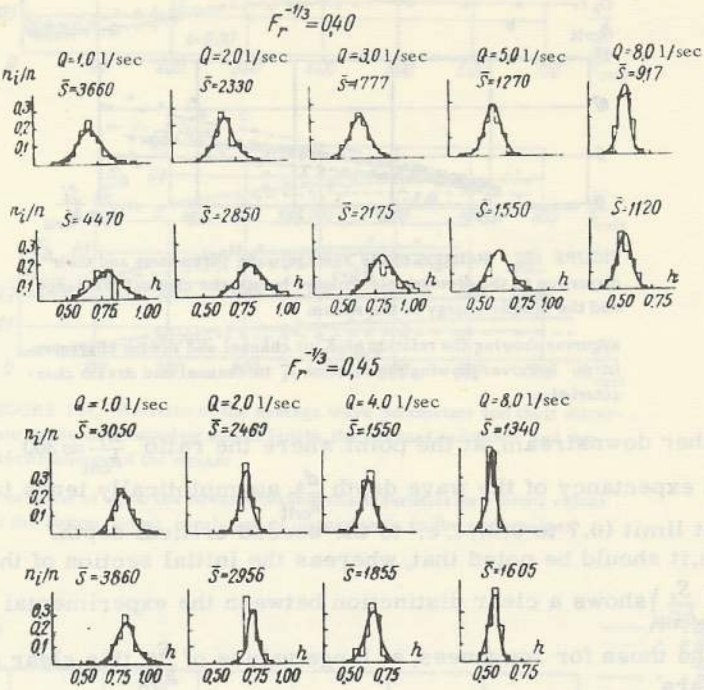


FIGURE 158. Probability curves of wave-depth distribution

The problem of the effect of aeration on wave motion, owing to its specific features, is dealt with in another report.

THE MODELING OF RIVER-CHANNEL PROCESSES BY MEANS OF MORPHOMETRIC ANALYSIS

Research Team: A. G. Nazaryan, Candidate of Technical Sciences, Senior Research Worker
M. S. Pokhsraryan, Junior Research Worker
M. I. Ter-Astvatsatryan, Junior Research Worker

The purpose of this study was to investigate the character of the changes in the longitudinal profile of a river as a result of a continuous lowering of the level of Lake Sevan, and to obtain the data required for the design of river-regulating structures.

One way of solving this problem is by tests on river-channel models in the laboratory.

When using models for channel-forming processes three independent parameters are used: the morphometric relation, Manning's general formula, and the movability of the erosible channel bottom.

The morphometric relation, advanced by M. A. Velikanov, relates the width ratio $\frac{b}{d}$ of a stream to its basic parameters: discharge q, average diameter of soil particles d, and the slope of the channel bottom i:

$$\frac{b}{d}=\alpha\left[\frac{q}{d^2\sqrt{gdi}}\right]^x. \qquad (1)$$

The numerical values for α and x were found by processing the field data for the rivers of the Lake Sevan basin (Figure 159).

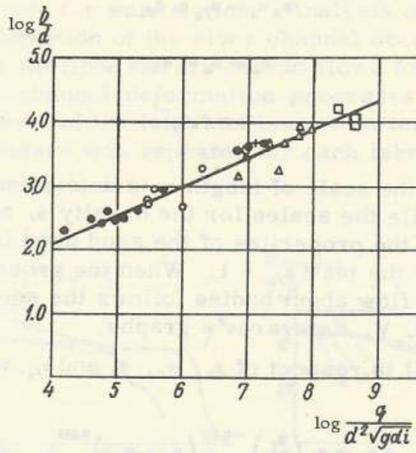


FIGURE 159. Graph for determining the numerical values of α and x

Manning's general formula connects the average flow velocity u_{av} with the slope i and the depth h, as well as with the average diameter d of soil particles:

$$u_{av}=\beta\left(\frac{h}{d}\right)^n\sqrt{ghi}, \qquad (2)$$

where n = an exponent; according to Goncharov $n=\frac{1}{6}$;

β = a constant coefficient; according to Goncharov β = 6.25.

The movability of the channel bottom is given a dimensionless expression:

$$\frac{h_i}{\sigma d}=f_0, \qquad (3)$$

where σ = the relative density as given by

$$\sigma = \frac{\rho_1 - \rho}{\rho} = \frac{\gamma_1 - \gamma}{\gamma}.$$

where ρ_1 and γ_1 = density and bulk weight of river load;

ρ, γ = idem of water;

f_0 = resistance coefficient of movable (erosible) river bottom.

Let us write equations (1) and (2) as scale ratios with the corresponding indexes, taking into account also the two obvious relations

$$q = u_{\text{av}} \cdot b \cdot h \text{ and } i = \frac{h}{a}.$$

we thus obtain:

$$\alpha_b = \alpha_d^{-0.25} \alpha_q^{0.5} \alpha_h^{0.25} \alpha_i^{-0.25}.$$

$$\alpha_u = \alpha_d^{-0.167} \alpha_h^{0.657} \alpha_i^{0.5}.$$

$$\alpha_h \alpha_i = \alpha_{f_0} \alpha_\sigma \alpha_d. \tag{4}$$

$$\alpha_q = \alpha_u \alpha_b \alpha_h.$$

$$\sigma_i = \alpha_h \alpha_i^{-1}.$$

It is assumed that the scale of length α_l is determined by the conditions of the laboratory, while the scales for the density α_σ and the diameter α_d are taken as a function of the properties of the sand used in the model. For the self-modeling area of the test: α_{f_0} = 1. When the process does not occur in the area where the flow about bodies follows the square law, the value for α_{f_0} is found from I. V. Egiazarov's graphs.

Solving equation (4) in respect of α_b, α_d, α_σ and α_{f_0}, we obtain:

$$\alpha_b = \alpha_l \left(\frac{\alpha_l}{\alpha_d}\right)^{-0.167} \left(\alpha_\sigma \cdot \alpha_{f_0}\right)^{0.833}.$$

$$\alpha_h = \alpha_l \left(\frac{\alpha_l}{\alpha_d}\right)^{-0.5} \left(\alpha_\sigma \cdot \alpha_{f_0}\right)^{0.5}. \tag{5}$$

$$\alpha_u = \alpha_l^{0.5} \left(\frac{\alpha_l}{\alpha_d}\right)^{-0.416} \left(\alpha_\sigma \alpha_{f_0}\right)^{0.583}.$$

$$\alpha_q = \alpha_l^{2.5} \left(\frac{\alpha_l}{\alpha_d}\right)^{-1.088} \left(\alpha_\sigma \alpha_{f_0}\right)^{1.916}.$$

The following actual values for the prototype river are entered into equations (1) to (4):

$$d = 0.002 \text{ m}; \ h = 0.60 \text{ m}, \ b = 35.0 \text{ m},$$

$$q = 20 \text{ m}^3/\text{sec}, \ i = 0.003, \ u_{\text{av}} = 0.95 \text{ m/sec}, \ \gamma = 2.65 \text{ t/m}^3.$$

For practical reasons the longitudinal scale of the model was taken as $\alpha_l = 100$ and the characteristics of the model material were

$$d = 0{,}8 \text{ mm and } \gamma_n = 2{,}20 \text{ t/m}^3.$$

All the other parameters of the model were calculated from equation (5).

After these preliminary calculations the models of the Argichi River, a tributary of Lake Sevan, was built in the open-air area of the Institute laboratory.

The model was 60 m long and 10 m wide and simulated the region of the lake including the mouth of the Argichi River. Tuffaceous sand was used as bottom for the river and the lake. The topography corresponded to conditions in the year 1933 (before the lowering of the lake level).

Before the beginning of the tests the model basin (lake) was filled with water up to the required level and a quantity of water corresponding to the river discharge, was made to flow through the model river.

The water level of the lake was lowered in accordance with the schedule, allowance being made for wave action. Analysis of the field data had shown that the main deformation of the river channel occurs during flood water. In the model tests the flood and dry-season flows for a given lake level were continued until the channel-deformation processes were approximately stabilized. The level of the lake was lowered after recession of the flood water. This procedure was repeated for each lake level.

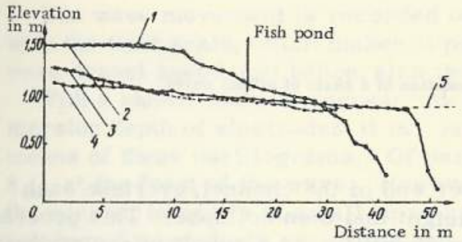


FIGURE 160. Results of model-river tests for determining the longitudinal profile of the river after lowering the base level

1-initial lake level; 2-initial lake bottom; 3-profile of the river bottom after lowering the lake level by 12 m; 4-model profile of river bottom after the lowering of the lake level by 12 m; 5-lake level in 1958.

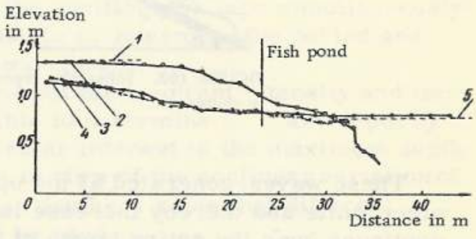


FIGURE 161. Results of model-river tests for determining the longitudinal profile of the river after lowering the base level

1-initial lake level; 2-initial lake bottom; 3-profile of the river bottom after lowering the lake level by 12 m; 4-model profile of river bottom after the lowering of the lake level by 12 m; 5-lake level at a different date.

Figures 160 and 161 show the results of model tests for determining the longitudinal profile of the model river after lowering the base level. The figures also give the actual measured longitudinal profile before and after lowering the base level.

As can be seen, the model and the natural profiles show a fairly close agreement.

INSTITUTE OF POWER ENGINEERING AND HYDRAULICS OF THE ACADEMY OF SCIENCES OF THE ARMENIAN S. S. R. DEPARTMENT FOR HYDRAULICS OF STREAMS AND STRUCTURES

Head: Professor A. K. Ananyan, Doctor of Technical Sciences

EXPERIMENTAL TECHNIQUE USED IN INVESTIGATING ROLLING WAVES IN CHUTES

Research Team: A. O. Gambaryan,
N. N. Mailyan

Observations on steep canals showed that at a certain ratio of the kinetic energy of the stream to the channel [hydraulic] resistance, the streamflow acquires a shooting character, in which movement is only possible in the form of a continuous chain of rolling waves (Figure 162).

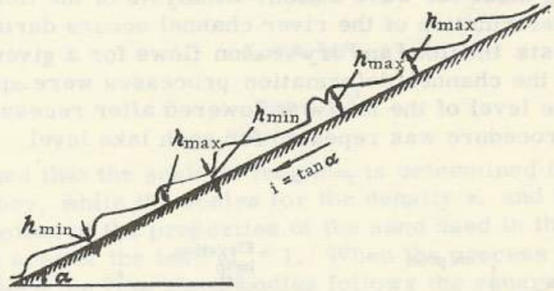


FIGURE 162. Schematic representation of a chain of rolling waves in chutes

These waves, generated at the upper end of the channel, overtake each other, unite and thereby increase in height, and then collapse. This process continues over the entire length of the channel. As a result of redistribution of discharge and flow velocity within each separate wave, the main part of the discharge, exceeding the average discharge at the upper end of the channel, is concentrated at the frontal zone of the wave.

These investigations, which are still being continued, are intended to find the relationship between the parameters of rolling waves (maximum depth h_{max}, wave length λ, wave-travel velocity, and wave discharge) and the stream and channel characteristics (slope, roughness, discharge, etc.) These relationships will provide the necessary data for the correct design and operation of chutes with due allowance for the existence of rolling waves.

In view of the high travel velocity of the waves, the electrode method of recording was used. The transducers of the recording device were made of thin streamlined lamellar electrodes immersed in the stream. The test data were then recorded on an oscillograph. Figure 163 shows the circuit diagram of the test and recording setup, and Figure 164 shows an example of an oscillogram.

The electrode transducers E, installed at five points along the model, ensured a synchronous tape recording of the wave travel.

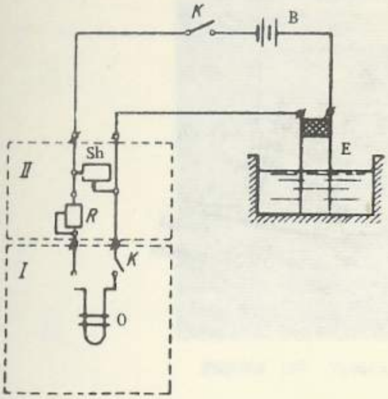


FIGURE 163. Circuit diagram of the wave-length measuring (and recording) unit

I-oscillograph; II-resistor block; E-electrode transducers; B-storage battery; K-simple (or knife) switch; Sh-shunting resistance; R-series-connected resistor; O-oscillator (loop of oscillograph).

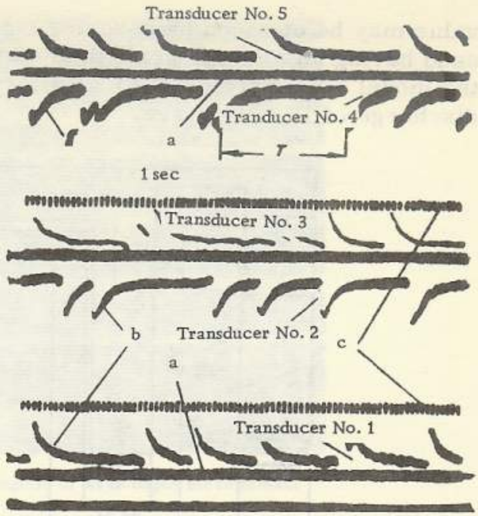


FIGURE 164. Typical oscillograms for wavelength measurements

a-arbitrarily chosen datum line; b-wave-profile line; c-time scale.

Time interval between neighboring points, 0.125 sec.

The wave movement is recorded on the oscillograph tape simultaneously with the time scale, which makes it possible to determine the period and wave-travel speed, and hence, also the wavelength.

With a known scale of corres' 'dence between current intensity and immersion depth of electrodes, it is ,ossible to determine t. ave depth by means of these oscillograms. Of particular interest is the maximum depth h_{max} at the front of the wave. However, in view of the nonlinear variation of the current intensity at small immersion depths, a somewhat different scheme of wave-depth recording had to be used.

This scheme (see Figure 165) consists in immersing in the stream a set of electrodes whose ends are placed at different distances from the channel bottom.

During their travel the waves "attack" those electrodes whose ends are below the wave crest. At this moment the transducers emit signals which are recorded on the oscillograph paper or film.

Figure 166 shows the general view of the unit, and Figure 167 shows types of oscillograms.

This method of depth recording is more convenient, as it does not require any linearization of the oscillator circuits and gives measurements of sufficient accuracy.

Flow velocities were measured by means of special miniature noncontact current meters (Figure 168) designed and manufactured at the Institute by M. Khachiyan.

The experiments were carried out with the following parameters of the model testing unit: model-bottom slope ($i = \tan\alpha$) 0.1; 0.15; 0.30; roughness coefficient of channel bottom and sides 0.011 and 0.015 (the second

value may be obtained by covering the freshly painted surface with quartz sand having an average grain diameter of 2-3 mm. The discharge through the model varied from 1.0 to 8.0 liter/sec; a few tests were carried out at a discharge of 16.0 liter/sec.

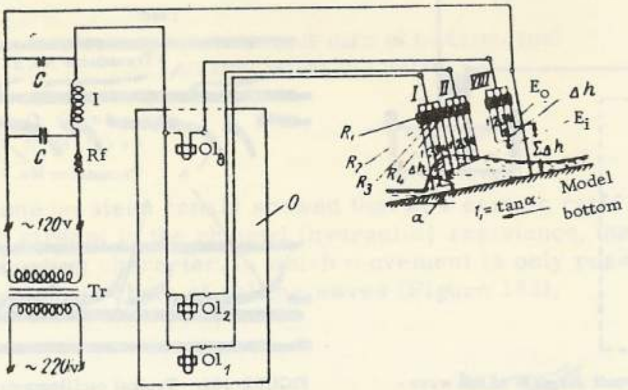


FIGURE 165. Circuit diagram of the wavelength signaling and measuring unit

Tr-step-down transformer; Rf-rectifier; C-capacitor; I-inductance coil; O-oscillograph; Ol-oscillators(loops); E_o-current-supply electrode; E_i-current-removal electrode; R-constant resistors in the circuit of each electrode.

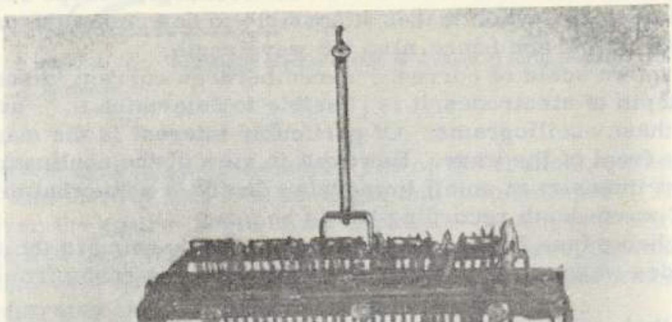


FIGURE 166. General view of transducer for wave-depth recording

The length of the model unit varied from 30 m (at i = 0.30) to 60 m (at i = 0.10).

Between 500 and 600 waves were measured in each test. The large number of observations, and the random nature of the phenomena, permitted the use of statistical methods of analysis.

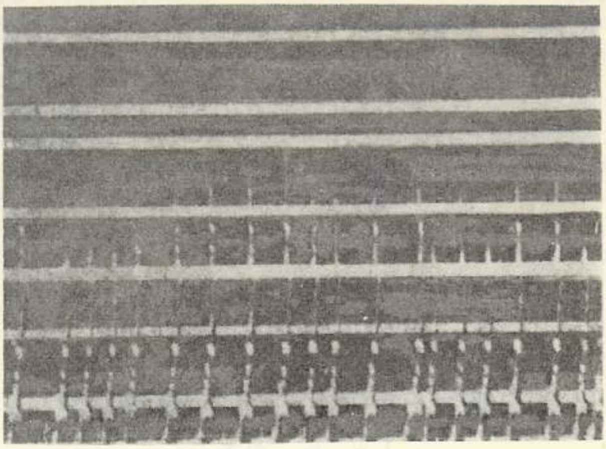


FIGURE 167. Typical oscillogram of wave-depth measurement

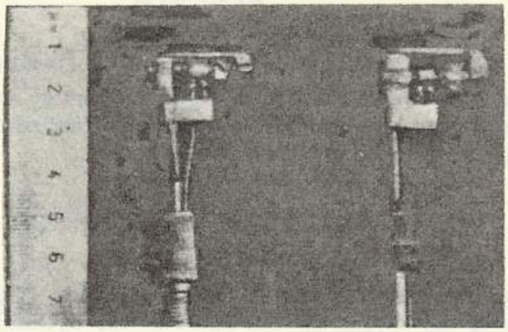


FIGURE 168. General view of noncontact miniature current meters

The results of the tests show that the waves formed in the chutes do not constitute trains of waves of equal parameters.

ON THE PROBLEM OF SUSPENDED-LOAD TRANSPORT BY TURBULENT STREAMS IN OPEN CHANNELS AND IN PRESSURE PIPES

Responsible for Research: V. G. Sanoyan, Candidate of Technical Sciences, Senior Research Worker

The aim of this study was to determine, both theoretically and experimentally, the degree of turbidity in open channels and in pressure pipes, and to study the effect of suspended load on the dynamics of a clear (nonturbid) streamflow

The differential equations for flow continuity and momentum written in vectorial form are

for solid phase

$$\frac{\partial S\bar{V}_s^*}{\partial t} + \operatorname{div} \bar{S} \left(\vec{V}_s^* \vec{V}_s^*\right) + \operatorname{div} \bar{S} \left(\vec{V}_s{}' \ \vec{V}_s{}'\right)^* - \frac{1}{\rho_s} \bar{S} \ \operatorname{div} \bar{P} -$$

$$- \frac{1}{\rho_s} R - SF_s{}^* = 0, \tag{1}$$

$$\frac{\partial \bar{S}}{\partial t} + \operatorname{div} \left(\bar{S}\vec{V}_s{}^*\right) = 0; \tag{2}$$

or for liquid phase

$$\frac{\partial\left(1-\bar{S}\right)\vec{V}^*}{\partial t} + \operatorname{div} \left(1-\bar{S}\right)\left(\vec{V}^*\vec{V}^*\right) +$$

$$+ \operatorname{div}\left(1-\bar{S}\right)\left(\vec{V}'\vec{V}'\right)^* - \frac{1}{\rho}\left(1-\bar{S}\right) \operatorname{div} \bar{P} +$$

$$+ \frac{1}{\rho} R - \left(1-\bar{S}\right)\bar{F}^* = 0, \tag{3}$$

$$\frac{\partial\left(1-\bar{S}\right)}{dt} + \operatorname{div}\left(1-\bar{S}\right)\vec{V}^* = 0. \tag{4}$$

The asterisks in these equations designate average values of the liquid volume; asterisks together with the subscripts, designate the volume of suspended sediment determined from formula

$$f^* = \frac{f(1-S)}{1-S}, \ f_s{}^* = \frac{\overline{fS}}{\bar{S}};$$

R is the vector of the forces of hydraulic resistance of the solid particles to the movement of the liquid, all the other letter symbols are those commonly used.

Let us plot equations (1) to (4) on a system of artesian coordinates x_1, x_2 where x_1 is parallel to the channel bottom and x_2 normal to x_1. In this case we obtain for a steady, uniform streamflow the following system of equations:

$$0 = \frac{d}{dx_2}\left[\rho_s \bar{S}\left(V_{s_1}{}' V_{s_2}{}'\right)^*\right] - R_1 + \rho_s \bar{S} g_i, \tag{5}$$

$$0 = -\frac{d}{dx_2}\left[P_s \bar{S}\left(V_{s_1}{}'^2\right)^*\right] - \frac{\bar{S}\partial\bar{P}}{dx_2} - R_2 - \rho_s \bar{S} g, \tag{6}$$

$$0 = -\rho \frac{d}{dx_2}\left[(1-\bar{S})(V_1{}' V_2{}')^*\right] + R_1 + \rho\left(1-\bar{S}\right) g_i, \tag{7}$$

$$0 = -\rho \frac{d}{dx_2}\left[(1-\bar{S})(V_2{}'^2)^*\right] - (1-\bar{S})\frac{\partial P}{dx_2} +$$

$$+ R_2 - \rho\left(1-\bar{S}\right) g. \tag{8}$$

This is not a closed system. To close the system we assume that, for small degrees of turbidity, the pressure is governed by the hydrostatic law; moreover, we assume that the distribution of a silt-laden stream differs but little from the distribution to a homogeneous stream.

If we introduce the unknown functions $F_1(\bar{S})$, $F_2(\bar{S})$, $F_3(\bar{S})$, which depend on the turbidity and on the particle diameter, and which take account of the influence of the solid particles on the characteristics of the streamflow, and expand them to a power series, we obtain (limiting the system to two terms)

$$(V_2'^2)^* = \alpha\, U_{\max}^2 \Phi_1(S) \approx x U_{\max}^2 (1 - K_1 \bar{S}), \tag{9}$$

$$(V_{s2}'^2)^* = \alpha\, U_{\max}^2 \Phi_2(S) \approx \alpha\, U_{\max}^2 (1 - K_2 S), \tag{10}$$

$$(V_1' V_2')^* = g_i (H - y) \Phi_3(\bar{S}) = -g_i (H - y)(1 - K\bar{S}). \tag{11}$$

If we transfer to dimensionless values

$$x_1 = xH,\ x_2 = yH,\ (V_{s_1}' V_{s_2}')^* = (U_s' V_s')^* g_i H,\ (V_{s_1}'^2)^* = \\ = (V_s'^2)^*\ U_{\max}^2, \tag{12}$$

then, according to (9)-(11), the system (5)-(8) will be

$$g_i \rho_s \frac{d}{dy} S\, (U'_s V_s')^* + R_x - \rho_s S g_i = 0, \tag{13}$$

$$\frac{\alpha U_{\max}^2}{H} \rho_s \frac{d}{dy} S\,(1 - K_2 S) + Sg\,(\rho_s - \rho) + Ry = 0, \tag{14}$$

$$\rho g_i \frac{d}{dy}(1 - S)(1 - KS)(1 - y) - Ry + \rho\,(1 - S)\, g_i = 0, \tag{15}$$

$$\alpha\rho\, \frac{U_{\max}^2}{H} \frac{d}{dy}(1 - S)(1 - K_1 S) - Ry = 0 \tag{16}$$

(here and below the signs designating averaged values are omitted).

Adding equations (14) and (16) and transforming, we obtain

$$B_s \frac{dS}{dy} - A\,\frac{dS}{dy} - S = 0, \tag{17}$$

$$A = \frac{\alpha U_{\max}^2}{gn}\left[\frac{\rho_s - (1 + K_1)\rho}{\rho_s - \rho}\right],$$

$$B = \frac{2\alpha\, U_{\max}^2}{gn}\left[\frac{K_1 \rho_s - K_2 \rho}{\rho_s - \rho}\right]. \tag{18}$$

the solution of equation (17)

$$B(S - S_0) - A \ln \frac{S}{S_0} = Y - Y_0, \tag{19}$$

gives S_0 – value of S at a depth Y_0.

The values of the coefficients K_1 and K_2 can be found by tests. Figure 169 presents in a $Y-Y_0$ and $\frac{U^2_{max}}{gn}\log\frac{S}{S_0}$ coordinates system, the results of Vanoni's test, worked out by the authors.

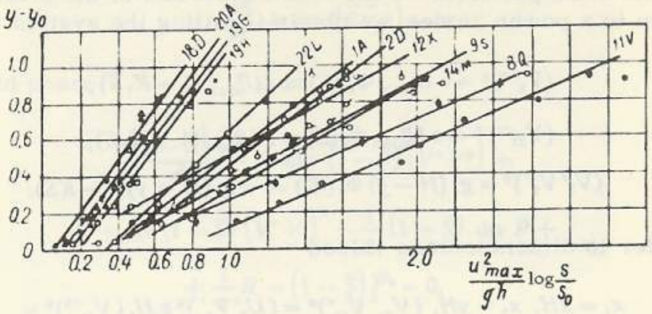


FIGURE 169. Results of Vanoni's test

Figure 170 shows the variation of coefficient K_1 with the fall velocity [also called sinking velocity of settling rate].

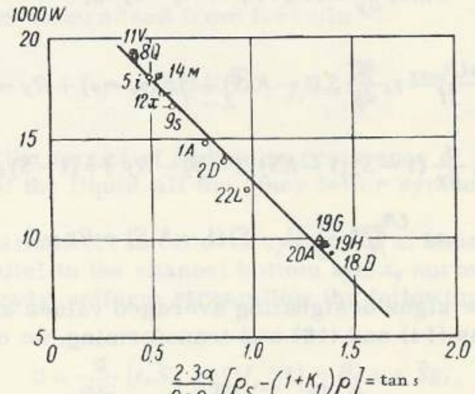


FIGURE 170. Variation of coefficient K_1 with the fall velocity [of suspended load]

So far there are no reliable experimental data for the determination of the numerical value of K_2. Therefore, the Institute is now carrying out special experiments involving high-speed motion-picture photography and photoelectric recording for streams of small turbidity. Similar methods permit the solution of the problem of hydraulic transport through pressure pipelines. For this purpose let us project the equation system (1)-(4) on spherical coordinates (Figure 171). Here, the Z axis coincides with the center line of the pipe, r is the radial coordinate and φ the azimuth. Omitting all the intermediate steps and transformations for a steady and uniform flow, we obtain the following system of equations for the solid load:

$$\frac{\rho}{r}\left[\frac{\partial S\,(V_z'\;V_\varphi')_s^*}{\partial\varphi}+\frac{\partial r^2 S\,(V_\varphi'\,V_r')_s^*}{\partial r}\right]+\frac{S\partial P}{\partial Z}R_z=0, \tag{20}$$

$$\frac{\rho_s}{r^2}\left[\frac{\partial}{\partial\varphi}Sr\,(V_\varphi'^2)_s^*+\frac{\partial r^2 S\,(V_\varphi'\,V_r')_s^*}{\partial r}\right]+\frac{S}{r}\frac{\partial P}{\partial\varphi}-$$
$$-R\varphi-\rho_s SF_{\varphi s}^*=0. \tag{21}$$

$$\frac{\rho_s}{r}\left[\frac{\partial}{\partial\varphi}\,S\,(V_r'\;V_\varphi')_s^*+\frac{\partial r S\,(V_z'^2)_s^*}{\partial r}-S(V_\varphi'^2)_s^*\right]+\frac{S\partial P}{\partial r}-$$
$$-R_z-SF_{rs}^*=0, \tag{22}$$

and a similar system for the liquid phase. The continuity equations turn into an identity. In order to close the system of equations (20)-(22) we proceed from assumptions similar to those made in solving the two-dimensional problem

$$(V_z'\;V_r')^*=V_*^2 r\,(1+K_1S), \tag{23}$$

$$(V_z'\;V_r')_s^*=V_*^2 r\,(1+K_2S), \tag{24}$$

$$(V_z'\,V_\varphi')^*=a\,V_*^2\frac{\partial S}{\partial\varphi}, \tag{25}$$

$$(V_z'\;V_\varphi{}')_s{}^*=a_1\,V_*^2\frac{\partial S}{\partial\varphi}. \tag{26}$$

Assumptions (25) and (26) are based on the fact that, in view of the symmetrical movement with regard to the vertical diametral cross section, the magnitudes $(V_z'\,V_\varphi')^*$ and $(V_z'\,V_\varphi')_s^*$ change in sign for angles $\pm\varphi$ (Figure 172). On the other hand, if $S=0$, these magnitudes become nil. The assumptions in (25) and (26) satisfy these conditions.

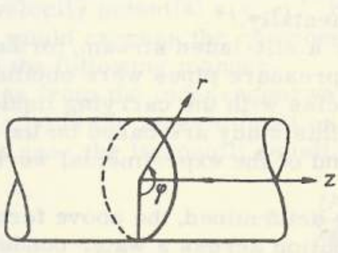


FIGURE 171. Projection of equations (1)-(4) on cylindrical coordinates

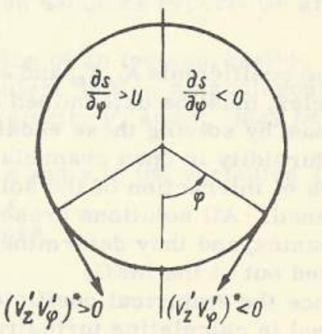


FIGURE 172.

Inserting formulas (23)-(26) in (20)-(22) and the similar equations for the liquid phase, we obtain, after linearization, the following differential equation:

$$\frac{a}{r}\frac{\partial^2 S}{\partial\varphi^2}+\sigma r\frac{\partial S}{\partial r}+2\sigma S-d=0, \tag{27}$$

where
$$\sigma=\frac{(1+K_2)\,\rho_s-(1+K_1)\,\rho}{\rho},\quad d=\left(2+\frac{V_{av}^2}{2V_*^2}\frac{\partial P}{\partial Z}\right)=$$
$$=2\left[1-\frac{\left(\frac{\partial P}{\partial Z}\right)_s}{\left(\frac{\partial P}{\partial Z}\right)_1}\right]. \tag{28}$$

The general solution of (27) is

$$S=\sum_{\lambda=1}^{\infty}A_\lambda\frac{1}{r^2}e^{\frac{-\lambda^2 a}{\sigma r}},\quad \cos\lambda\varphi-\frac{d}{2\sigma}. \tag{29}$$

The coefficients R_λ are determined from the given distribution of turbidity along the pipe periphery, i.e.

$$S=S_0(\varphi)\ \text{for}\ r=1. \tag{30}$$

According to (30) we have

$$S=\sum_{\lambda=1}^{\infty}\frac{2}{\pi}\left(\frac{1}{r}\right)^2 e^{\frac{\lambda^2 d}{\sigma}\left(\frac{1}{r}-1\right)}\cos\lambda\varphi\int_0^{\pi}S_0(\zeta)\cos\lambda\zeta\,d\zeta-\frac{d}{2\sigma}. \tag{31}$$

From these equations it becomes now easy to determine the components of the forces of resistance of the solid load to the movement of the liquid. For the longitudinal component we obtain

$$R_z=\frac{\rho_s V_*^2}{r_0}\left(2S+r\frac{\partial S}{\partial r}\right)+S\frac{\partial P}{\partial Z}\frac{\rho V_{av}^2}{2r_0}. \tag{32}$$

The coefficients K_1, K_2, and a, which depend on the diameter of the solid particles, must be determined experimentally.

Thus, by solving these equations for a silt-laden stream, formulas for flow turbidity in open channels and in pressure pipes were obtained and the forces of interaction of the solid particles with the carrying liquid were determined. All solutions presented in this study are based on the laws of mechanics, and they determine the trend of the experimental work now being carried out at the InEG.

Once the empirical coefficients are determined, the above formulas can be used in calculating turbidity distribution across a water conduit, taking into account the effect of solid particles on the dynamics and the kinematics of the flow. The method may also be used for determining the difference in flow velocity between the solid and liquid particles at different points of the turbulent stream.

ON THE TWO-DIMENSIONAL PROBLEM OF SPREADING (FLATTENING) OF A FLASHY STREAM OF INCOMPRESSIBLE LIQUID

Responsible for Research: G. A. Babadzhanyan, Candidate of Physics and Mathematics

The problem of the two-dimensional flow of an incompressible liquid in an open channel consists in solving the following system of equations:

$$V_x \frac{\partial V_x}{\partial x} + V_y \frac{\partial V_x}{\partial y} = -g \frac{\partial h}{\partial x},$$
$$V_x \frac{\partial V_x}{\partial x} + V_y \frac{\partial V_y}{\partial y} = -g \frac{\partial h}{\partial y}, \tag{1}$$
$$\frac{\partial}{\partial x}(hV_x) + \frac{\partial}{\partial y}(hV_y) = 0,$$

where $V_x(x, y)$ and $V_y(x, y)$ = components of the velocity vector along axes x and y;

$h(x, y)$ = depth of stream.

These equations were obtained assuming:

1) a hydrostatic law for the pressure distribution along the stream depth;

2) the flow-velocity vector to be constant along the stream depth;

3) the positive bottom slope to be equal to the inclination of the friction forces.

For a steady streamflow the constant energy and the absence of eddies reduce the original system of equations to a single equation

$$\left(1-\frac{V_x^2}{gh}\right)\frac{\partial^2 \varphi}{\partial x^2} - 2\frac{V_x V_y}{gh}\frac{\partial^2 \varphi}{\partial x \partial y} + \left(1-\frac{V_y^2}{gh}\right)\frac{\partial^2 \varphi}{\partial y^2} = 0. \tag{2}$$

For flashy streams ($V > \sqrt{gh}$) this equation becomes hyperbolic and may be solved by the method of characteristics.

The study investigates the turbulent motion of an incompressible liquid with no velocity potential $\varphi(x, y)$. For turbulent motion an analogous function that would express the components of velocity V_x and V_y may be obtained in the following manner.

We pass from the independent variables x and y to the variables ξ and ψ where $\psi(x, y)$ is the stream function and $\xi = x$.

In this case the Bernoulli equation becomes

$$h + \frac{U^2}{2g} = G_0(\psi), \tag{3}$$

where $G_0(\psi)$ = function of the stream function $\psi(x, y)$.

Resorting to the equations of motion and continuity we obtain after certain transformations (introduction of new variables, differentiation, and some algebraic calculations) the relationship between the velocity components V_x, V_y and function $f(\xi, \psi)$ which satisfies a definite differential equation.

This relationship has the following form:

$$V_y = -\frac{\partial f}{\partial \psi}, \; V_x = \sqrt{V^2 - \left(\frac{\partial f}{\partial \psi}\right)^2}. \tag{4}$$

The equation for determining function $f(\xi, \psi)$ has the following form:

$$g\frac{dG_0(\psi)}{d\psi}\frac{\partial f}{\partial \psi}\sqrt{\frac{g}{2}}\left(\frac{\partial f}{\partial \xi}\right)^{-1}\frac{\partial^2 f}{\partial \xi}\left[3\sqrt{\frac{g}{2}} + \frac{1}{2}\left(\frac{\partial f}{\partial \xi}\right)^{-1/2}\left(\frac{\partial f}{\partial \psi}\right)^2 -\right.$$

$$\left. - gG_0(\psi)\left(\frac{\partial f}{\partial \xi}\right)^{-1/2}\right] + 2g\frac{\partial^2 f}{\partial \psi^2}\left[G_0(\psi) - \sqrt{\frac{2}{g}}\left(\frac{\partial f}{\partial \xi}\right)^{1/2}\right] +$$

$$+\sqrt{2g}\left(\frac{\partial f}{\partial \xi}\right)^{-1/2}\frac{\partial f}{\partial \psi}\frac{\partial^2 f}{\partial \psi \partial \xi}. \tag{5}$$

Here $G_0(\psi)$ is the given function of ψ.

After determining function $f(\xi, \psi)$ from this equation, we can find the velocity components V_x and V_y from (4) and then the relationship between the stream depth h and function $f(\xi, \psi)$

$$h = \sqrt{\frac{2}{g}\frac{\partial f}{\partial \xi}}. \tag{6}$$

With the known function $f(\xi, \psi)$ we can determine the flow lines in the plane x, y.

As can be seen, equation (5) for determining function $f(\xi, \psi)$ is complex and its general solution cannot be obtained.

This equation has been investigated for three particular cases.

1. The stream depth h depends only on the stream function, i. e. $h = h(\psi)$.

In this case the unknown parameters V and h can be found from the following equations:

$$\frac{d}{d\psi}\frac{V^2}{(ghh')^2} = \frac{2}{gh^2h'},$$

$$V^2 = 2gG_0(\psi) - 2gh.$$

With a given shape of function $G_0(\psi)$ we can solve this system and then obtain the law of variation of velocity V and depth h.

In this case the flow lines form a family of circles with a common center.

2. Constant depth of stream (h = const).

In this case the problem may be solved by using the following equations:

$$\frac{\Theta'(\psi)}{\sqrt{U^2 - \Theta'^2}} = \text{const},$$

$$U^2 = 2gG_0 - 2gh,$$

where $H'(\psi)$ is an arbitrary function of ψ. The resulting flow lines become parallel straight lines.

3. Laminar (nonturbulent) motion ($G_0(\psi)$ const). In this case the above method leads to the same results obtained by other investigators (G. I. Sukhomel, I.A. Shnepkov) who solved this problem by a more complicated method.

INSTITUTE OF POWER ENGINEERING AND HYDRAULICS OF THE ACADEMY OF SCIENCES OF THE ARMENIAN S. S. R. DIVISION FOR FIELD INVESTIGATIONS

Head: B. I. Bek-Marmarchev, Candidate of Geographical Sciences

DETERMINATION OF THE SOLID LOAD CARRIED BY RIVER TRIBUTARIES INTO LAKE SEVAN

Research Team: B. I. Bek-Marmarchev, Candidate of Geographical Sciences
V. N. Khamagortsyan, Candidate of Technical Sciences

The drastic lowering of the lake level causes bottom and bank erosion of the lake's tributaries and an increase in the amount of the load carried into the lake.

Owing to the similarity of the physiographical conditions along the whole of the lake shore, it was sufficient to determine the load volume in the Argichi, Tsakkar, and Bakhtak rivers, and to use the data as a basis for all other tributaries.

The volume of solid load was determined by direct measurements of the alluvial cone at the mouth of the river. Since most of the load is carried into the lake during the flood period, measurements were made before and after the floods. During this period the stage variations were continuously recorded at a gaging station located 2-3 km upstream from the river mouth.

River depths were measured at previously fixed sites by means of echo soundings which made it possible to obtain a sufficiently exact profile of the river bottom. The soundings made by the PEL-1 echo sounder, measured depths of up to 6-8 m and covered an area at about 500×500 m at the mouth of the river. The echo sounder was mounted on a YaL-4 launch which moved at a speed of 5 to 7 km/hour. The position of the boat was determined by two theodolites set up on the river bank at a distance of 600 to 800 m from each other, and the echo soundings were recorded on a tape. The river profiles were measured at intervals of 20 to 50 m.

The amount of river load transported by the flood was determined by comparing the river profiles obtained before and after the flood passage.

Soil samples of the river-bottom deposits near its mouth were taken before and after the floods to study size grading and petrographic composition.

From aerial photographs of the river-mouth area, carried out especially for these investigations, it was found that the shore line of the lake changes with the lowering of the lake level, assuming gradually the shape of a curve jutting out into the lake. This change is particularly noticeable if one compares the actual position with the bathymetric map drawn in 1930 (before the lake level was lowered). The convex shape of the shore lines indicates the development of an alluvial cone. These lines (also called rolls), which

are the result of the action of wind-driven waves and correspond to the level of the lake in different years, are clearly visible on the aerial photographs.

The volume of solid load was determined by comparing the 1:10,000 bathymetric map of 1930 with the topographic map to the same scale based on the 1957 survey.

By this (integral) method of determining the volume of solid load, one can follow the development of the erosion and accumulation processes on an exposed area over a time interval of 27 years.

The data obtained by measuring the alluvial cone (i. e. the load volume) for one flood period as well as by the integral method for a period of 27 years, were compared and used in determining the total annual amount of solid load carried into Lake Sevan by its tributaries.

MIIVKh IMENI V. R. VIL'YAMS CHAIR FOR UTILIZATION OF WATER-POWER RESOURCES

Head: Professor D. Ya. Sokolov, Doctor of Technical Sciences

HYDRAULIC LABORATORY INVESTIGATIONS OF NONSTEADY OPERATION OF HYDRO UNITS AT THE IRKUTSK HEP DURING DROP OF LOAD

Scientific Guidance: Professor D. Ya. Sokolov, Doctor of Technical Sciences
I. I. Kovalenko, Candidate of Technical Sciences, Lecturer

Responsible for Research: P. E. Tkachenko, Candidate of Technical Sciences

Research Team: A. F. Gubin, N. A. Palishkin, V. P. Shkarin and V. A. Shchetinin, Engineers

Hydro units of run-of-river power plants are protected against runaway conditions, caused by a sudden failure of the automatic regulators, by quick-dropping intake gates installed as a rule in the headwater section at the inlet to the turbine casings. Since these inlets are usually large, the intake gates must be made of corresponding size which increases their cost.

Recently, some run-of-river plants have been equipped with quick-dropping gates installed at the tailwater side, either at the outlet of the draft tube or within the tube at the beginning of the flared section of the draft tube. Since the draft-tube gates are much smaller than the intake gates at the turbine casing, their cost is much lower.

The absence of theoretical data on gate-closing conditions, as well as lack of experience in the design and operation of such gates, made it necessary to carry out detailed laboratory research and to verify the results in the field. Practice shows that the emergency lowering of quick-dropping draft-tube gates sometimes causes the turbine rotor to rise. This is due to the fact that when the gates are lowered rapidly the upward axial pressure of the streamflow on the runner exceeds the combined dead weight of the moving parts and the downward axial pressure of the stream flowing through the turbine.

At the Irkutsk HEP the draft-tube gates are installed at the initial portion of the draft-tube enlarged section, their over-all sizes being only a third of the size of the intake gates mounted at the turbine-casing inlet.

The main purpose of the laboratory tests was to determine the conditions for the lowering of the quick-dropping gates, and hence, to reduce runaway of the hydro turbines and the dangerous upward axial pressure of the stream on the turbine runner. The problem was solved by studying the sequence of the following features of nonsteady operation during different closing schedules of the gates:

a) nature of the variation of the rotational speed, torque, and axial pressure of the stream on the turbine runner;

b) nature of the variation of the water pressure in front of and behind the runner and the gates;

c) nature of the variation of the water discharge through the turbine.

Apart from these investigations, the study involved visual observations on other characteristic phenomena and on the turbine behavior during nonsteady operation caused by the closing of the quick-dropping gates.

These investigations were carried out on a 1:28.8 model of one Irkutsk HEP turbine set. The turbine model, made of plexiglass, was installed in a "mirror" flume of about 14 m length. Dynamic tests were conducted on a turbine of the PL-487-25 type [adjustable-blade turbine according to Soviet designation of turbine types] and consisted in recording the variations of the nonsteady operation during the movement, at different closing speeds, of the quick-dropping gates. Static tests were conducted on a PL-577-25 turbine, and consisted in recording and plotting nonsteady processes of operation at a stationary position of the quick-dropping gates. In the first case the gates were installed at the draft-tube outlet, in the second, they were established at the initial portion of the flared section of the draft tube.

The model unit was equipped with all the necessary measuring instruments, their readings being recorded by means of a cathode-ray oscilloscope.

The following measurements were made: rotation speed of the turbine, torque, axial pressure of the water stream on the turbine wheel, stream pressure in front of the wheel and gates, and behind them, the movement of the gates, and the water discharge through the turbine. In dynamic tests, in which these parameters varied with the amount of gate movement, the readings of all measuring instruments were recorded on oscilloscopes. In static tests, conducted at a certain definite (stationary) position of the gate, only the amount of axial stream pressure on the runner was recorded on the oscilloscope; all other readings were taken directly from the instruments.

These investigations yielded data characterizing the behavior of the hydraulic unit under nonsteady operation conditions and under different conditions of movement of the quick-dropping gates. The variation of the axial stream pressure on the runner and of the rotational speed were found to depend on the speed of gate closing. The higher this speed, the greater the negative axial stream pressure and vice versa. On the other hand, the critical runaway speed decreases with the increase in the speed of gate lowering. The permissible speed of lowering the quick-dropping gates ensuring the safe operation of the hydraulic units lies between definite limits. If the draft tube is divided into compartments by means of splitter piers, each gate should be closed at a different uniform speed: one, at a relatively high speed (30-40 sec), the second, at a lower speed (for 120 or more seconds). This complicates the design of the gate-control mechanisms and gate operation.

The conclusion drawn from the laboratory tests was that correctly planned quick-dropping gates installed in draft tubes may be used as an emergency protection against runaway conditions in case of a failure in the automatic regulator. Further experimental data will contribute to the solution of the problem of optimum closing conditions for this type of gates.

The investigation results have been sent to various production organizations,(Mosgidep, Lengidep, MISI imeni V. V. Kuibyshev), and to the management of the Irkutsk HEP for field research. These results may also be used at other power plants having the same type of gates and turbines.

NIMI CHAIR OF HYDRAULICS AND THEORETICAL MECHANICS

Head: M. M. Skiba, Candidate of Technical Sciences, Lecturer

FREE HORIZONTAL SPREAD OF THE STREAMFLOW IN THE TAILWATER OF HYDRO STRUCTURES

Responsible for Research: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research by: S. K. Kuznetsov, Senior Engineer

By free horizontal spread of a streamflow we mean a movement in which the water mass in the tailrace does not affect the spread of the upper part of the spilling jet.

The investigation of this type of flow is useful for the study of the hydraulic processes developing in the tailwater at the beginnnig of the flow; for the selection of the best shape of the chute outlets; for a better understanding of the factors leading to a flashy flow in the case of sudden widening of the river channel, etc.

The length of the experimental concrete flume was 600 cm, its width 100 cm, and its height 50 cm. The tests involved measurement of discharges, depth, and flow velocity. The direction of currents was determined by means of silk threads, dye and small metal balls.

Figure 173 shows the pattern of the free horizontal spread of a water stream.

The whole stream was divided arbitrarily into three regions:

A – inside, leaf-shaped, spreading region;

B – outside region separated from A by bottom rollers;

C – region at the corner of the hydro structure.

The nature of the flow in regions A and C has been described by I. P. Linchevskii whose studies are accepted as very reliable. On the other hand, opinions are divided on the nature of flow in region B.

The results of our investigations showed that in region B a series of bottom rollers are formed. The upstream rollers form a boundary which separates regions A and B.

The bottom rollers have a rotational and translational [helical] movement. With the increase in angle θ the translational movement decreases whereas the rotational movement increases (Figure 173, a).

The Froude number for various points of region B, calculated from flow-velocity measurements, did not exceed 4. The liquid mass coming from

region A flows over the bottom rollers, inducing a wave motion, and changes the direction of the velocities toward the stream axis.

In region B the directions of the bottom and surface currents coincide.

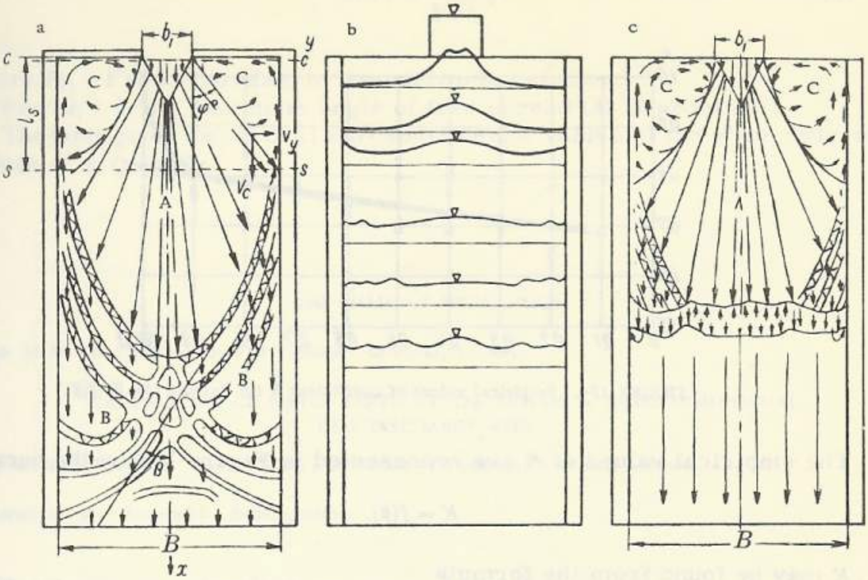


FIGURE 173. Free horizontal spread of a streamflow

Thus, region B constitutes the zone of junction between the flashy stream of region A and the calm stream in the tailrace. At the junction there is a series of slanting wave-jumps in which the bottom rollers are analogous to the rollers of slanting wave-jumps in the two-dimensional problem.

As the tailwater level increases, a hydraulic jump with a surface roller forms gradually at the end of region A. The tests showed no stable surface rollers in region B, when the displacement wave ceased.

As the perfect jump in region A and the slanting wave-jumps in region B move upward, there arrives a moment when, at the upper portion of B, the displacement wave, owing to the very small flow velocities, cannot be stopped and the water mass collapses upstream, filling the spreading zone C.

The sheetlike shape of the spreading stream is modified abruptly, which leads to the appearance of either a flashy stream or a circulating current passing into the tailwater.

The maximum natural depth at which free spread is still possible (Figure 173, c) is found from formula

$$h_2 = 0.5\beta \cdot Kh_c\left[\sqrt{1+8\,\frac{Q^2B}{gb_1^3\,h_c^2}}-1\right], \tag{1}$$

where b_1 = width of spillway bay (outlet opening);

B = total width of tailwater channel;

Q = discharge through the outlet opening;

h_c = depth in the contracted section;

$\beta = \frac{b_1}{B}$;

K = empirical coefficient allowing for the nonuniform distribution of velocity and discharge across the river section.

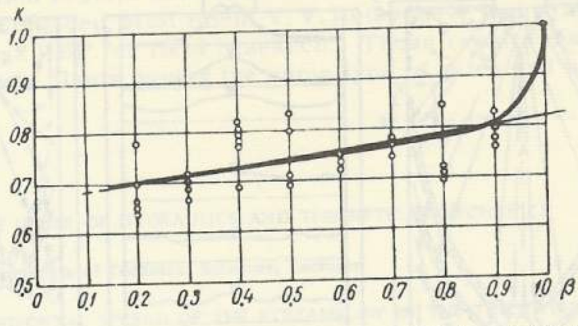


FIGURE 174. Empirical values of coefficient K (in formula (1); $K=f(\beta)$

The empirical values of K are represented in Figure 174 by the curve

$$K=f(\beta).$$

K may be found from the formula

$$K=0.8-(0.9-\beta)0.15. \tag{2}$$

For $\beta < 0.2$ experiments show that the free horizontal spread of the stream is possible as long as the tailwater depth does not exceed the depth in the contracted section.

TABLE 1

b_1, cm	B, cm	Q, l/sec	Fr_1	l_s, cm	tan φ, experim.	tan φ, calculated	Deviation, %
20	100	3.12	3.24	57	0.70	0.78	+11.3
20	100	7.73	24.2	125	0.32	0.32	0
20	100	11.12	50.7	176	0.22	0.20	− 9.1
30	100	5.12	3.88	50	0.70	0.71	+ 1.4
30	100	9.03	14.3	92	0.38	0.37	− 2.6
30	100	14.9	40.4	145	0.24	0.22	− 8.3
40	100	8.78	7.3	60	0.50	0.52	+ 4.0
40	100	13.89	19.1	95	0.316	0.322	+ 3.0
40	100	21.17	48.2	140	0.214	0.203	− 5.0
50	100	9.549	5.34	46	0.54	0.60	+11.0
50	100	15.72	15.6	80	0.31	0.35	+12.0
50	100	18.5	20.2	90	0.28	0.31	+10.0

The angle of free spread (Figure 173) can be determined from formula

$$\tan\varphi = \frac{1.41}{\sqrt{\mathrm{Fr}_1}}, \tag{3}$$

where Fr_1 = Froude number in the contracted section.

For $\mathrm{Fr}_1 = 1$, the maximum angle of free spread (3) is about 55°.

The table gives theoretical and experimental values of $\tan\varphi°$ for different values of β, Q and Fr_1.

NIMI CHAIR OF HYDRAULICS

Head: M. M. Skiba, Candidate of Technical Sciences, Lecturer

CALCULATION OF WATER DEPTH AT THE END OF A THREE-DIMENSIONAL FREE DISCHARGE WEIR

Responsible for Research: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research by: Ovcharenko, Junior Lecturer

Handbooks of hydraulics recommend that the depth at the end of a submerged weir be considered as critical and determined by the formula

$$h_{\mathrm{cr}} = \sqrt[3]{\frac{\alpha Q^2}{g b^2}}. \tag{1}$$

This formula was proposed by B. A. Bakhmetev at the beginning of this century, for flat broad-crested weirs, assuming a calm and laminar flow over the weir. If, however, there is a sudden contraction of the stream over the weir, this assumption no longer holds and formula (1) cannot be used. Since there is no existing formula which applies to the case of a sudden contraction of the stream, we decided to investigate this problem experimentally. In the course of this investigation the following two factors were determined: (a) deformation of the cross-sectional area of the stream at the end of the weir, as a function of the lateral contraction and the different parameters of the intake channel; (b) the relationship governing the water depth at the end of the weir in the case of sudden contraction of the stream section. The investigations were conducted on a model described in this book in the annotation "On the Calculation of the Free Surface of Deformed Streamflows", which also contains a description of the experimental technique used. The tests were divided into four series; each series covered three groups of tests. Figure 175 shows the curves attained in the first test series. The experimental data show a marked nonuniformity in the water depth at the end of the weir. Thus, the difference between the depth at the lateral wall and the depth along the stream axis is on the average 25 to 30%. The test results also point to the important influence of inertia forces. Figure 175

gives the experimental points in coordinates ν and K. Here $K = \frac{h_{av}}{H}$. A detailed analysis of the curves $K = f(\nu)$, based on these experimental points, showed that these curves may be described as square parabolas

$$K = K_0 + A(\nu - \nu_0)^2, \tag{2}$$

where ν_0 is that value of ν at which function $K = f(\nu)$, has a minimum value, i.e. K_0.

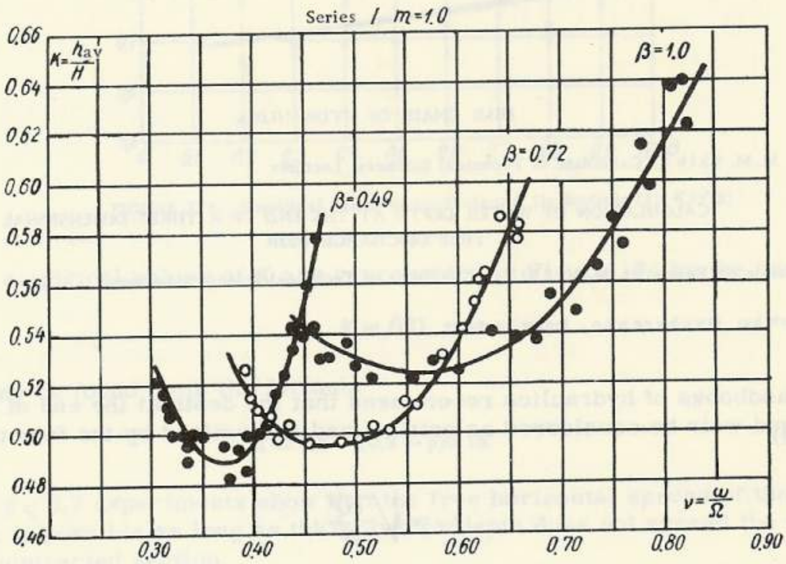


FIGURE 175. Curves obtained in the first test series

Table 1 gives the parameters of function $K = f(\nu)$ for all test series.

TABLE 1

m \ β	0.49			0.72			0.1		
	ν_0	K_0	A	ν_0	K_0	A	ν_0	K_0	A
1.0	0.370	0.488	10.0	0.475	0.494	2.82	0.560	0.524	1.57
1.25	0.350	0.473	8.4	0.440	0.448	3.35	0.505	0.500	1.60
1.50	0.315	0.486	4.5	0.410	0.487	2.75	0.475	0.515	1.85
1.75	0.310	0.474	7.4	0.385	0.490	2.55	0.425	0.493	1.65

The values of ν_0 and K_0 in this table were taken directly from the graph (Figure 175), A was found by trial and error. The curves $K = f(\nu)$ calculated from formula (2) and data in Table 1 show good agreement with the experimental points. For practical purposes the authors calculated $K = f(\nu)$ for $\beta = 0.5$-1.0 with intervals of 0.1 for different values of m. For $m = 1.0$, the curves $K = f(\nu)$ are shown in Figure 176. The calculation was made as

follows: resorting to the data in Table 1, we at first plotted the auxiliary curves $\nu_0 = f(\beta, m)$, $K_0 = f(\beta, m)$ and $A = f(\beta, m)$ (the last not shown here). These curves permitted us to find ν_0, K_0 and A for any β and m and thus made it possible to calculate the necessary curves by means of formula (2).

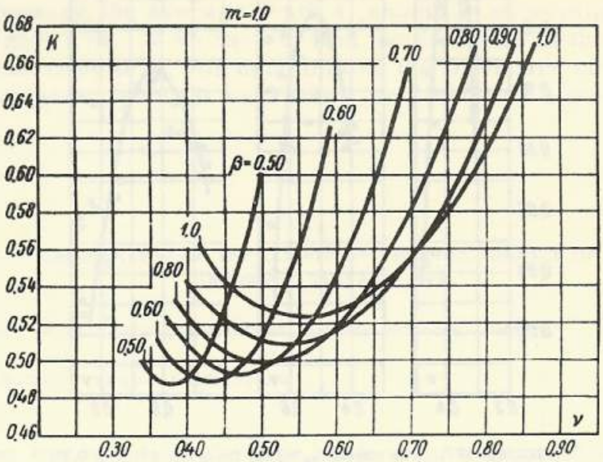


FIGURE 176. Curves $K = f(\nu)$ for $m = 1.0$

Let us now consider coefficient α_h which allows for the nonuniform distribution of stream depths. Using the formulas given in the annotation "Calculation of Curves of Free Surface of a Deformed Streamflow" and also experimental data, we determined α_h for the whole test series and plotted the graph shown in Figure 177. An analysis of function $\alpha_h = f(\nu)$, shows that it can be described by formula

$$\alpha_h = \alpha_{h_0} - a(\nu - \nu_0)^2. \tag{3}$$

For this given case ν_0 and α_{h_0} are the coordinates of the peak of function $\alpha_h = f(\nu)$. The curves for $m = 1.0$ obtained from (3) are plotted in Figure 177. The values for ν_0, α_{h_0} were taken from the graphs, the parameter a was found by trial and error. The values for ν_0, α_{h_0} and a are given in Table 2.

TABLE 2

β / m	0.49			0.72			0.1		
	ν_0	α_{h_0}	a	ν_0	α_{h_0}	a	ν_0	α_{h_0}	a
1.0	0.380	0.155	13.0	0.510	0.125	4.50	0.620	0.108	1.75
1.25	0.360	0.152	15.0	0.470	0.133	3.75	0.580	0.116	1.35
1.50	0.340	0.172	14.5	0.430	0.134	2.60	0.520	0.138	1.95
1.75	0.320	0.158	19.9	0.420	0.140	2.75	0.480	0.134	2.20

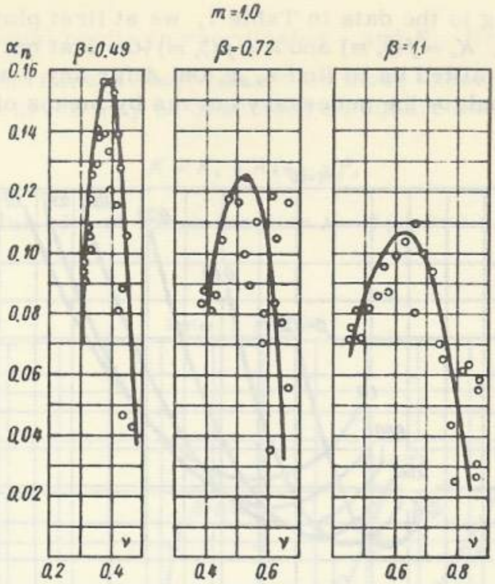


FIGURE 177. The curves $\alpha_h = f(\nu)$ for $\beta = 0.49$; $\beta = 0.72$ and $\beta = 1$ at $m = 1.0$

The value of α_h can be found by formula (3) for any value of β and m, by means of the parameters ν_0, α_{h_0} and a given in Table 3. This table was constructed with the aid of auxiliary curves $\nu_0 = f(\beta, m)$, $\alpha_{h_0} = f(\beta, m)$ and $a = f(\beta, m)$, while these latter were derived from Table 2.

TABLE 3

m \ β	0.50			0.60			0.70		
	ν_0	α_{h_0}	a	ν_0	α_{h_0}	a	ν_0	α_{h_0}	a
1.0	0.390	0.153	12.0	0.448	0.138	7.25	0.502	0.127	4.80
1.25	0.367	0.151	14.0	0.416	0.142	7.00	0.462	0.134	4.15
1.50	0.346	0.172	11.0	0.387	0.150	3.95	0.425	0.136	2.75
1.75	0.326	0.158	15.0	0.377	0.147	4.40	0.414	0.141	2.95

m \ β	0.80			0.90			1.0		
	ν_0	α_{h_0}	a	ν_0	α_{h_0}	a	ν_0	α_{h_0}	a
1.0	0.550	0.118	3.40	0.588	0.112	2.40	0.620	0.108	1.75
1.25	0.508	0.127	2.60	0.547	0.121	1.70	0.580	0.116	1.35
1.50	0.462	0.131	2.25	0.493	0.131	2.10	0.520	0.138	1.95
1.75	0.443	0.131	2.50	0.464	0.134	2.30	0.480	0.134	2.20

Summarizing the above results, the following points should be noted:

1. If the streamflow at a weir contracts suddenly, the distribution of depths at the end of the weir is nonuniform, a fact pointing to the action of

inertia forces. Then, Bakhmetev's formula based on the assumption of a calm (slowly changing) streamflow is not applicable.

2. It seems practical to adopt as the theoretical stream depth at the end of the weir, the average depth h_{av}, and to express the nonuniform distribution of depth by a nonuniformity coefficient α_h.

3. If necessary, the average depth h_{av} should be determined from the graphs in Figure 176. If we have to find the deviation of the actual depth from the mean value (e.g. for determining the freeboard of the weir walls) we must calculate α_h, from (3) and Table 3, and then find σ from formula $\sigma = \alpha_h \cdot h_{av}$.

CALCULATION OF THE CURVES OF THE FREE SURFACE OF DEFORMED STREAMFLOWS

Scientific Guidance: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research by: I. Kh. Ovcharenko, Junior Lecturer

At present, the curves of the free surface [of a streamflow] are determined by means of the basic formula of nonuniform motion, derived for a condition of gradually changing motion. However, it frequently happens that the stream, flowing through a structure, becomes so deformed that this formula is no longer applicable. Such is the case of the flow through a steep chute with its entrance formed by return walls. Here, the stream entering the chute inlet contracts both in the horizontal and the vertical plane. Centrifugal forces acting along the cross-sectional area of the stream affect the hydraulic elements of the stream, and particularly the depth distribution. Three-dimensional contraction of a streamflow is a very complex phenomenon, which has been insufficiently studied so far. The authors have attempted to investigate this phenomenon experimentally for the case of rectangular steep chutes with the inlet section formed by return walls. The chute model was made of planed boards treated with linseed oil and eoated with enamel paint. The approach channel, of trapezoidal cross section, was made of the same material. The dimensions of the model were: 1) length of the approach channel L = 17.0 cm; 2) clear width of the chute $b = 14.4$ cm; 3) length of entrance section of the chute $\delta = 15.6$ cm; 4) length of the chute section $l = 28.5$; 5) slope of the chute $i = 0{,}105$; 6) slope of the approach channel and chute inlet $i = 0$.

The purpose of these investigations was to determine: a) the degree of deformation of the stream across the sections and the length of the chute as a function of the contraction at the entrance; b) the possibility of using the well-known integral equations of nonuniform flow in calculating the depth of water in the chute.

During the investigations the head H varied from 2 to 17 cm and the discharge Q from 1.0 to 16.0 liter/sec. The inclination coefficients of the slopes of the approach channel were: $m = 1.0$, 1.25, 1.50, and 1.75; the ratio $\beta = \frac{b}{b_c} = 0.49$, 0.72, and 1.0; and the degree of lateral contraction $\nu = \frac{\omega}{\Omega} = 0.2$ to 0.8. Here b_c = the bottom width of the approach channel; ω = cross-

sectional area of the stream at the beginning of the chute; and Ω = cross-sectional area of the stream before entering the chute. The discharge was measured by means of a previously calibrated triangular weir. The head both at the model and the triangular weir was measured by stationary level gages. The depth at the overfall sill was measured by a movable level gage, at points and measuring sites shown in Figure 178. Eighty tests were carried out during the whole research period.

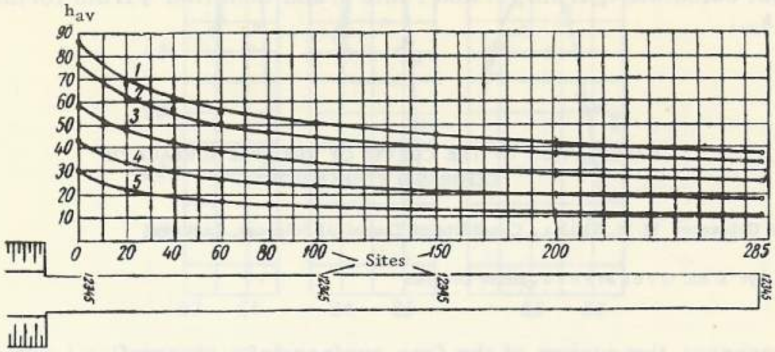


FIGURE 178. Graph of depth variation at the overfall sill

The investigations yielded the following results:

A. The streamflow deformation may be expressed by a coefficient of nonuniformity of depth distribution over the stream cross-section. This coefficient, α_h, can be expressed by the formula

$$\alpha_h = \frac{\sigma}{h_{av}}. \qquad (1)$$

The mean square deviation σ of the depth from the mean value is found from

$$\sigma = \sqrt{\frac{\Sigma (h_i - h_{av})^2}{n-1}}. \qquad (2)$$

where h_i = depth at the measuring point;

$h_{av} = \frac{\Sigma h_i}{n}$; n = number of measuring points.

Using the measured depths h_i we determined h_{av}, and then from formulas (1) and (2) the values of σ and α_h. Figure 179 shows some characteristic curves of variation of α_h along and across the stream, as a function of the contraction at the chute inlet. It can be seen that: a) the values of α_h vary from 0.02 to 0.18; b) at the end of the overfall the maximum values of α_h drop to 0.10. Thus, the depth over the cross-sectional area of the stream at the overfall differs from h_{av} by $\sigma = \alpha_h \cdot h_{av}$ within the limits for $\sigma = (0.02 - 0.18) h_{av}$.

B. The experimental curves for the free surfaces were compared with the theoretical curves plotted by M. M. Skiba's method. In plotting the theoretical curves we assumed as the starting depth h_{av}, the depth measured at

the beginning of the overfall sill. The roughness coefficient was found experimentally.

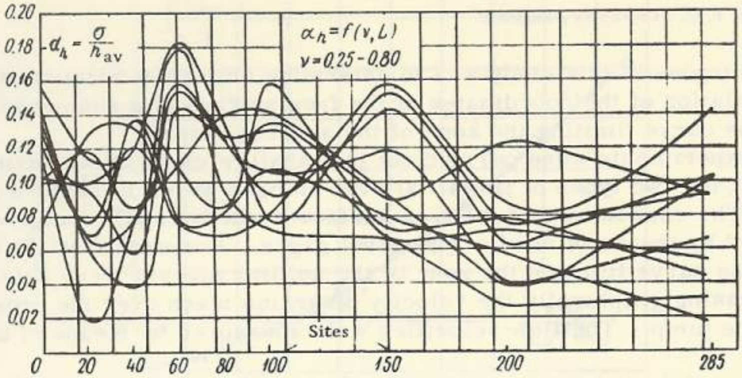


FIGURE 179. Curves of variation of a_h along and across the stream, as function of its contraction at the chute inlet

Figure 178 shows the theoretical curves of the free surface and the average depths h_{av} in cm, measured in several tests. The table below shows the deviation of the experimental average depths from the calculated values:

0–1 %	1–2 %	2–2.5 %	2.5–6 %
42	36	13	9

It can be seen that 78% of the average depth values show a deviation of only 0 - 2% from the theoretical values, whereas 22% of the average depth values show a deviation of more than 2% from the theoretical value.

C. The experimental investigations permit us to recommend the following method for calculating the depth at the sloping section of rectangular chutes with return-wall types of inlet:

1) the average depth h_{av} is found by the formula for nonuniform flow;

2) in determining the freeboard in the chute one should take into account that the deviation σ of the depth from its average values varies within the limits $\sigma = (0.02 - 0.18)\, h_{av}$;

3) the initial theoretical depths should be determined from graphs (see our preceding annotation "Calculation of Water Depth at the End of a Three-dimensional Free Discharge Weir").

PLOTTING THE CURVES OF THE FREE SURFACE OF SUBMERGED JUMPS AND THE CURVE LIMITING THE ZONE OF SPILLING WATER

Scientific Guidance: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research by: P. M. Stepanov, Engineer

The purpose of this study was to determine empirical relationships for the calculation of the coordinates of the free surface of a submerged jump, and of the curve limiting the zone of the spilling water.

The experiments were carried out in a shallow glass flume, having a width of 0.25 m, for two types of flows: a) over an ogee weir; b) under a flashboard. The coordinates of the free surface of the hydraulic jump were measured by means of needle-type level gages. For each test the coordinates of the curve limiting the zone of the spilling stream were determined by processing graphically the velocity diagrams taken over the cross section of the jump. The flow velocities were measured by means of a Pitot tube.

Since at the cross sections the boundaries of the interface between the spilling zone of the jump and the zone above it are formed by curves orientated with their convex side downward, it has been necessary to calculate the average coordinate in order to plot the curves limiting the zone of the spilling stream in each cross section.

In deriving the relationships we proceeded in the following manner: we derived the equation for the coordinates of the curves of free surfaces, and the equation connecting the full stream depth H at the given cross section of the jump with the depth h_{sp} of the spilling stream measured at the same cross section.

The curves of the free surface of a submerged hydraulic jump may be divided into two sections: a relatively small section of the curve orientated with its convex portion downward beginning from the barrier (the starting point of the roller), until the point of depth h_2 (i. e. the smallest depth of the jump); and the remaining section from the point of depth h_2 until the end of the jump (this portion of the curve is orientated with its convex side upward). As a rough approximation the first section of the curve may be taken as a straight line.

The relationship for determining the coordinates of the free surface was established for the second section of the curve. The analysis of the curves for the free surface of the hydraulic jump, obtained experimentally, shows that the basic factors which determine the shape of these curves are the submergence ratio ζ, the specific discharge q, and the Froude number in the contracted section of the stream.

The experimental points are fairly close to the curve (Figure 180) plotted by the trial and error method according to the coordinates

$$1-\frac{H-h_c}{h_n-h_c};\quad \frac{x\sqrt{h_c}}{\zeta^2 h_{cr}\sqrt{h_{cr}}}$$

where H = full depth at the given cross section of the jump;

h_n and h_c = respectively the natural and the contracted depth of the stream;

x = distance from the end of the jump;

h_{cr} = critical depth;
ζ = submergence ratio.

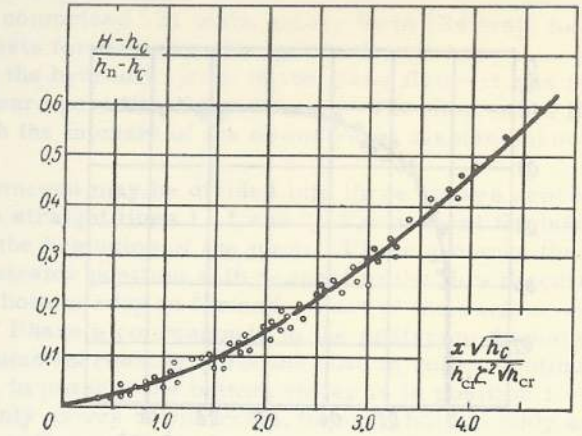


FIGURE 180. Graph for plotting the free surface of the second section of the curve for a submerged jump

This curve has the shape of a semicubic parabola.

By the method of "averaging errors" we obtained the equation of the curve

$$1-\frac{H-h_c}{h_n-h_c}=0.056\left(\frac{x\sqrt{h_c}}{\zeta^2 h_{cr}\sqrt{h_{cr}}}\right)^{3/2}. \tag{1}$$

Solving (1) with regard to H, we obtain

$$H=\left[1-x^{3/2}\frac{0.056\,(\sqrt{h_c})^{3/2}}{\zeta^3\,(\sqrt{h_{cr}})^{9/2}}\right](h_n-h_c)+h_c. \tag{2}$$

The factor

$$0.056\,\frac{(\sqrt{h_c})^{3/2}}{\zeta^3(\sqrt{h_{cr}})^{9/2}}=C \tag{3}$$

does not depend on x and is a constant value for given initial and final conditions. Designating it by C, and substituting in (2), we can write after suitable transformations

$$H=h_n+x^{3/2}(h_c-h_w)C. \tag{4}$$

By solving equation (4) we obtain the formula for determining the distance between the end of the jump and the given depth

$$x = \sqrt[3]{\left[\frac{H - h_n}{C\,(h_c - h_n)}\right]^2} \tag{5}$$

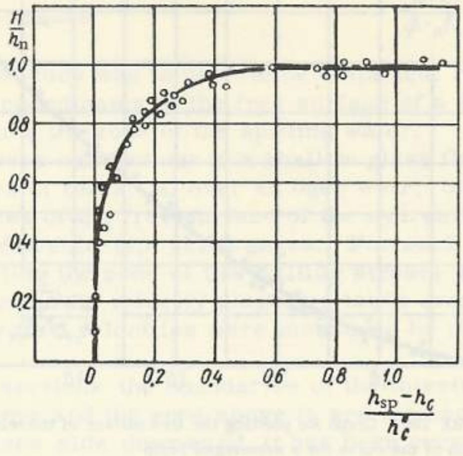


FIGURE 181. Graph of function $h_{sp}=f(H)$ for determining the depth of the spilling stream

Formula (2) was based on the results of 79 tests, in which the Froude number Fr_c ranged from 5 to 10 and the submergence ratio ζ from 1 to 2.2.

The depth of the spilling stream is determined from the graph $h_{sp}=f(H)$ shown in Figure 181.

This graph was plotted from the data of 17 tests, for which the coordinates of the spilling-stream curves were calculated.

LENGTH OF SUBMERGED HYDRAULIC JUMPS

Scientific Guidance: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research by: P. M. Stepanov, Engineer

The purpose of this study was to establish relationships, based on test data, for determining the length of a submerged hydraulic jump in a rectangular channel.

The tests were carried out in a glass flume, 0.25 m wide and 8.5 m long, for two types of outflow: a) flow over an ogee weir; and b) flow under a gate for a Froude-number range $Fr = (5 \text{ to } 50)$ and submergence ratios $\zeta = \frac{h_b}{h''_c} = (1 \text{ to } 2.5)$.

Geometric similarity of these two jumps was ensured by the equal values of ζ and Fr_c, and the tests were able to cover the range of cases most frequently encountered in practice.

The study comprised 172 tests, among them 134 tests for flow under a gate and 38 tests for flow over an ogee weir.

Observing the hydraulic jump in the glass flume, it was seen that a bottom eddy appears periodically at its end. The moment of its appearance coincides with the moment of the downstream displacement of the surface roller.

This phenomenon may be divided into three phases represented in Figure 182. The straight lines 1 - 1 and 2 - 2 represent the boundaries of displacement of the beginning of the jump. Phase a marks the beginning of the jump in the extreme position with respect to the flow direction. At the end of the jump a bottom eddy is formed. Part of the surface roller is forced downstream. Phase b corresponds to the upstream displacement of the roller, its volume increasing while the bottom roller continues to move downstream. In phase c the bottom roller is in position 1 - 1; at this moment, it suddenly moves downstream, forms a bottom eddy and expels part of the surface roller. The author did not notice any permanent bottom eddy.

Thus, at the end of the jump the flow direction of the stream changes continuously, which possibly accounts for the increased fluctuation in flow velocity and pressure which has been noticed by many investigators.

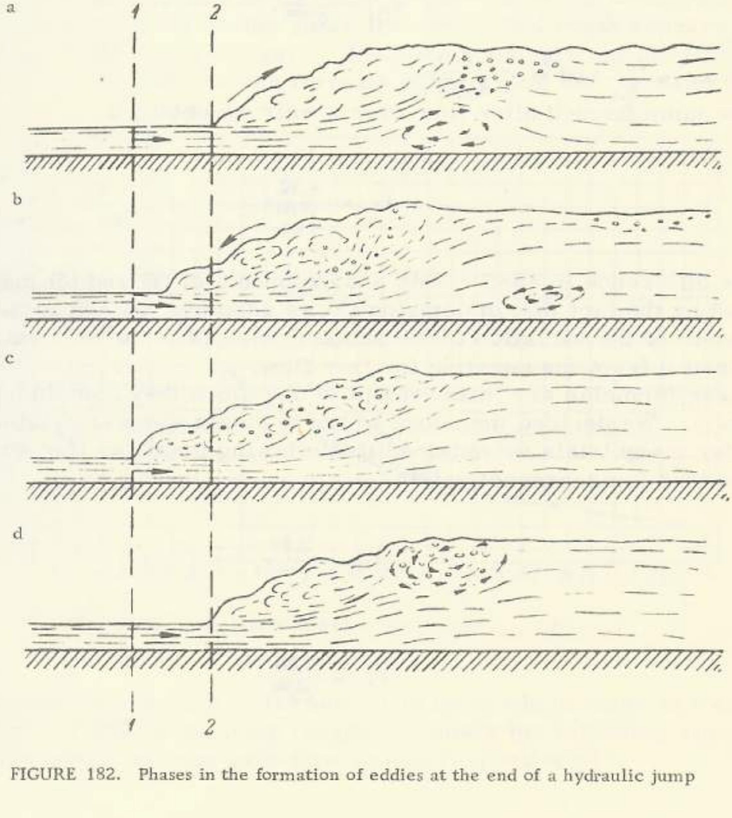


FIGURE 182. Phases in the formation of eddies at the end of a hydraulic jump

In these tests the length of the hydraulic jump was determined by measuring the length of the horizontal projection of the zone over the spilling stream. The position of the end of the jump was fixed visually by observing the flow of small shavings, threads and dyed water. The beginning of the jump was considered to be: for the flow under a gate, the gate; and for the flow over an ogee weir, the junction of the weir with the flume bottom.

The verification of the formula suggested by D. I. Kumin

$$l_{s.\,j.} = 6\,(h_s - h_c) \tag{1}$$

for determining the length of the submerged jump, showed that it gives values that deviate considerably from the experimental data. This deviation increases with the increase of the submergence ratio, attaining 15 to 18% for $\zeta = (2 - 2.5)$.

Analyzing the factors affecting the length of a submerged jump (q; h_c; h_b), the author used in the formula the parameters characterizing the following values: (Figure 183: critical depth $h_{cr} = f(q)$, the Froude number in the contracted section $\mathrm{Fr}_c = f(q : h_c)$ and the geometrical height of the jump $l_{s.\,j.} = (h_b - h_2) = f(h_c : h_b)$.

The following formula was obtained for the length of the hydraulic jump formed after flow under a gate:

$$\lambda_{s.\,j.} = \frac{6.46}{\zeta_{s.\,j.}^{0.885}}, \tag{2}$$

where $\lambda_{s.\,j.} = \frac{l_{s.\,j.}}{h_{cr}}$ and $\zeta_{s.\,j.} = \frac{l_{s.\,j.}}{h_{cr}\sqrt{\mathrm{Fr}_c}}$,
for the jump formed after flow over a weir we obtained

$$\lambda_{s.\,j.} = \frac{4.32}{\zeta_{s.\,j.}^{0.587}}. \tag{3}$$

The difference in coefficients and exponents of (2) and (3) may be explained by the fact that in deriving (3) we assumed the contracted depth, independent of the variation in the submergence ratio, to be constant; we determined it from the equation for free flow.

These formulas are inconvenient in use since they contain a fractional exponent. We decided therefore to try to obtain simpler relationships.

After many trials we obtained the following formulas (for a weir-type and a gate-type jump, respectively):

$$\lambda_{s.\,j.} = \frac{3.19}{\zeta_{s.\,j.}^{0.986}}, \tag{4}$$

$$\lambda_{s.\,j.} = \frac{2.88}{\zeta_{s.\,j.}^{0.986}}. \tag{5}$$

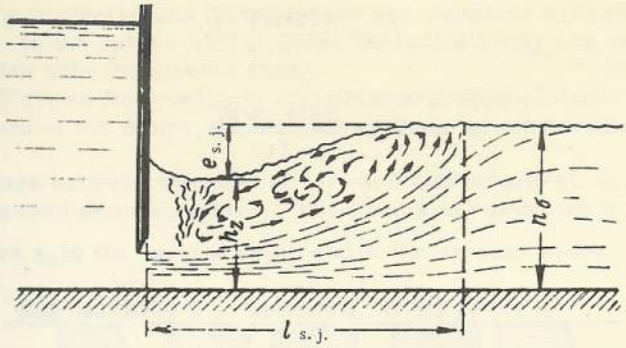


FIGURE 183.

In deriving formula (5) we took into account the variation in the contracted depth with a change in submergence ratio.

Figure 184 shows experimental points processed in coordinates $\lambda_{s.j.}$ and $\zeta_{s.j.}$ and the curve defining formula (4). As can be seen from Figure 184, the experimental points are fairly close to the curve. The difference between the coefficients in (4) and (5) is due to the technique of measuring the jump length. For a flow over a weir the length of the jump is shorter by $(2 - 2.5)\,h_c$, than the jump length for flow under a gate since, in the first case, the contracted depth appears at the point of junction of the weir with the flume bottom, while, in the second case, the contracted depth appears at a distance of $(2 - 2.5)\,h_c$ from the gate. Thus, when calculating the jump length for flow over a weir, we can use formula (4) reducing the result by $2h_c$.

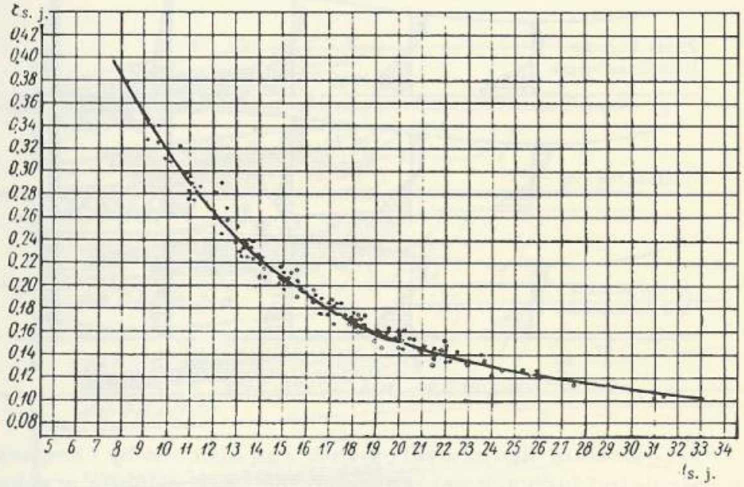


FIGURE 184. Function of hydraulic-jump length $\lambda_{s.j.} = f(\zeta_{s.j.})$

Rounding off the exponent in (4) and (5) to unity which leads to a certain increase (by 1 - 1.5%) in the jump length, we obtain the following simpler formulas (for weir-type and gate-type jumps, respectively):

$$l_{s.j.} = \frac{3.19 h_{cr} \sqrt{Fr_c}}{e_{s.j.}} \tag{6}$$

$$l_{s.j.} = \frac{2.88 h_{cr} \sqrt{Fr_c}}{e_{s.j.}}. \tag{7}$$

ON THE KINEMATIC STRUCTURE OF A SUBMERGED JUMP

Scientific Guidance: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research by: M.P. Stepanov, Engineer

The purpose of this work was to study the velocity distribution for a submerged hydraulic jump in a rectangular channel as well as the nature of its variation with a change in the submergence ratio.

The tests were carried out in a shallow glass flume 0.25 m wide and 8.5 m long for a Froude-number range $Fr_c = 5$ to 50, a submergence ratio $\zeta = 1$ to 2, and for two types of outflow: under a gate and over an ogee weir.

The flow velocities were measured by means of a Pitot tube at 4 to 5 sections within the jump region and at two sections downstream from the jump. At each of these sections the velocity was measured along three, and in some tests along eight verticals.

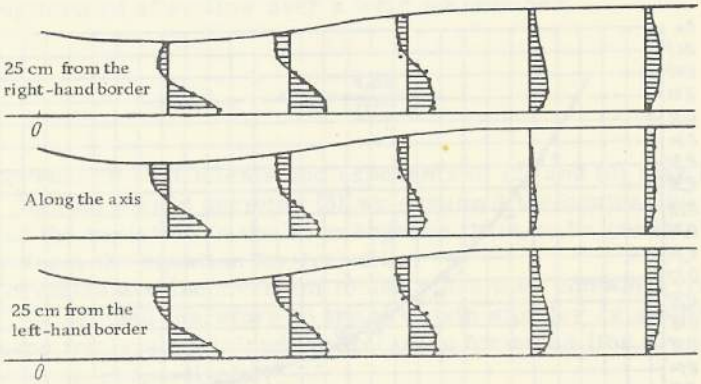


FIGURE 185. Flow-velocity graphs

The correctness of the measurements was checked by comparing the discharges calculated for each section from the flow-velocity graphs, with the discharge measurements made with the aid of a triangular weir. The discrepancy did not exceed ÷ 12% which indicates a sufficient accuracy of the tests.

Analyzing the flow-velocity distribution, we found that in the area of straightforward flow of the jump, and at a certain distance below it, the

distribution is characterized by two peaks symmetrical with respect to the stream axis. In the return-stream zone we noticed only one velocity peak which coincides with the stream axis.

Figure 185 shows flow-velocity distribution graphs plotted for three longitudinal sections of the jump; an analysis of these graphs confirms our conclusions.

The interface between the zone of forward and return-streams, forms a concave, downward oriented curve. The radius of curvature depends on the ratio $\frac{h_n}{b}$, where h_n is the natural depth and b the stream width.

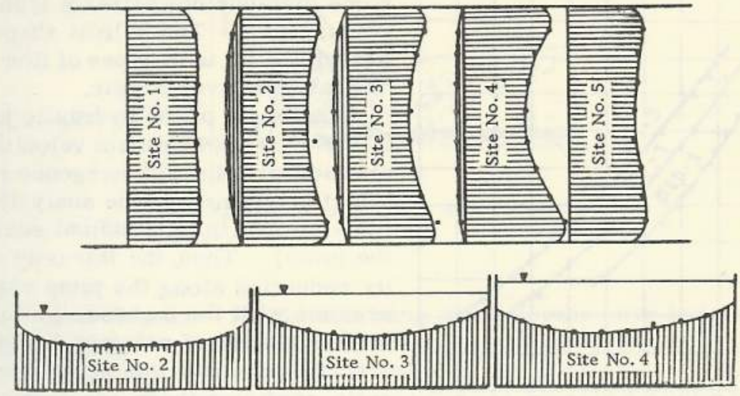


FIGURE 186. Graphs showing distribution of specific discharges of forward and return streams

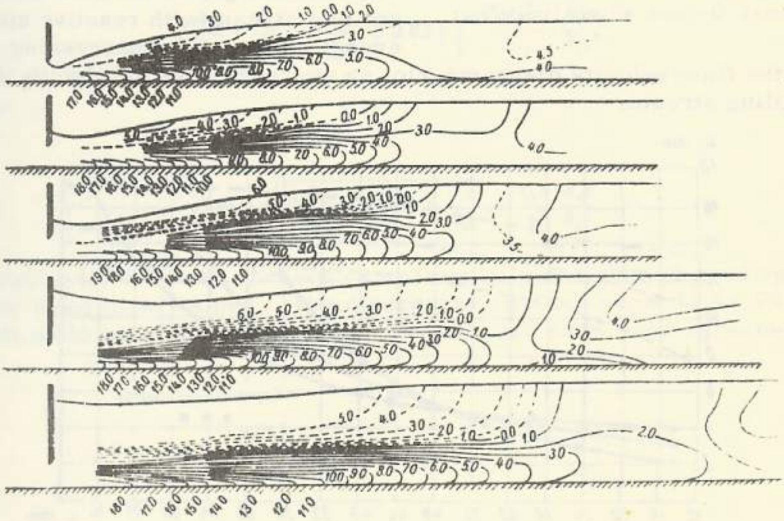


FIGURE 187. The effect of gate opening on the nature of flow-velocity distribution along the longitudinal section of the jump a = 0.4 dm; Q = 11.02 liter/sec; Fr = 13.1

The greater this ratio, the smaller the radius of curvature. Figure 186 shows the interfaces between a forward and return-stream; these interfaces were plotted from zero velocity points taken on the velocity-distribution graphs. The same figure also shows the graphs of specific discharges with two peaks.

These peaks shift gradually from the beginning of the jump toward its end where their relative value reaches its maximum.

The analysis of the isolines drawn through cross and longitudinal sections of the jump, fully confirms V. V. Vedernikev's conclusions on the helical shape of flow forming two branches in the region of the jump and for some distance downstream from it (Figure 187). The helical shape is characteristic of both types of flow: under the gate and over a weir.

The length of the hydraulic jump and of the zone of maximum velocities increases with the submergence ratio ζ, a fact confirmed by the analysis of isolines plotted in longitudinal sections [of the jump]. Thus, the intensity of velocity reduction along the jump region decreases with the increase in this ratio.

The graphs of velocity drop (see Figure 188) as a function of submergence ratio, confirms these conclusions.

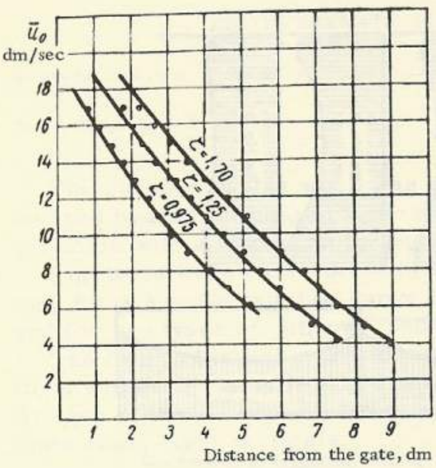


FIGURE 188. Reduction of flow velocity u_0 along the jump, as a function of the submergence ratio ζ; Q = const; a = const (axial plane)

If we conceive the hydraulic jump as consisting of two zones – a) the spilling zone of active kinetic energy with a constant discharge, b) the zone above the spilling stream with reactive kinetic energy – we can, by processing graphically the flow-velocity diagrams, plot an average curve enveloping the zone of spilling stream.

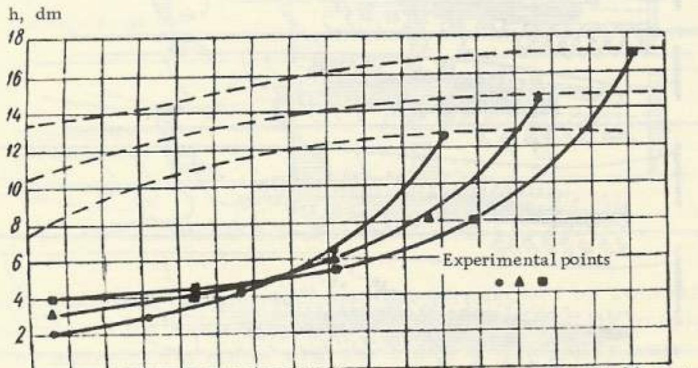


FIGURE 189. Curves showing the boundaries of the spilling stream. P, H, and Q = const (flow over a weir)

The analysis of these curves shows that in a submerged hydraulic jump formed by flow over an ogee weir, the initial depth of the spilling stream, the so-called contracted depth, varies with the submergence ratio $\left(\zeta = \frac{h_n}{h_c''}\right)$ Figure 189.

For practical purposes it is sometimes necessary to determine this depth as well as the minimum depth h_2 at the weir (Figure 190).

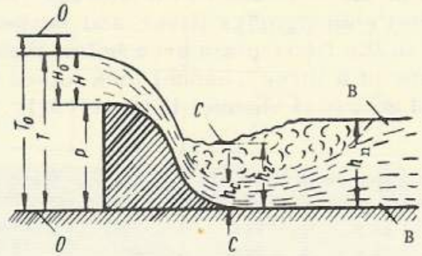


FIGURE 190. Minimum depth h_2 at the end of the weir

The relationship used for determining these depths have been obtained by jointly solving the Bernoulli equation for sections O-O and C-C, and the momentum equation for section C-C and B-B assuming that the distribution of the hydrodynamic pressure in section C-C is governed by the hydrostatic law, while the correction factor for momentum and kinetic energy $\alpha = \alpha_0 = 1$.

The equation for h_2 is

$$2\frac{h_{cr}}{h_n} + \frac{h_n^2}{h_{cr}^2} = \frac{h_2^2}{h_{cr}^2} + 2.82\varphi\sqrt{\frac{T_0}{h_{cr}} - \frac{h_2}{h_{cr}}}, \tag{1}$$

and for

$$\frac{T_0}{h_{cr}} = \sqrt{2\frac{h_{cr}}{h_n} - 2\frac{h_{cr}}{h_c} + \frac{h_n^2}{h_{cr}^2}} + \frac{0.5}{\varphi^2}\frac{h_{cr}^3}{h_c^2}. \tag{2}$$

Equation (1) has been checked experimentally and showed good agreement with the theoretical values. The discrepancy did not exceed ± 4 - 5% which permits us to conclude that equation (2) will also show good agreement.

SOME RESULTS OF THE INVESTIGATION OF THE NOVO-TROITSKOE CHUTE

Research Team: M. M. Skiba, Candidate of Technical Sciences, Lecturer
N. K. Shul'ga, Candidate of Agricultural Sciences, Lecturer
P. F. Kononenko, Candidate of Technical Sciences, Lecturer
I. Kh. Ovcharenko, Junior Lecturer

The chute of the Novo-Troitskoe hydro development in the Stavropol' Territory is located at the left bank of the storage reservoir. It discharges flood waters of the Bol'shoi Egorlyk River, and, between floods, supplies the needs of consumers in the flood-plain area below the reservoir. The chute spillway has the shape of a three-channel lock joined to the sloping section by a stilling pool and an outlet channel (Figure 191).

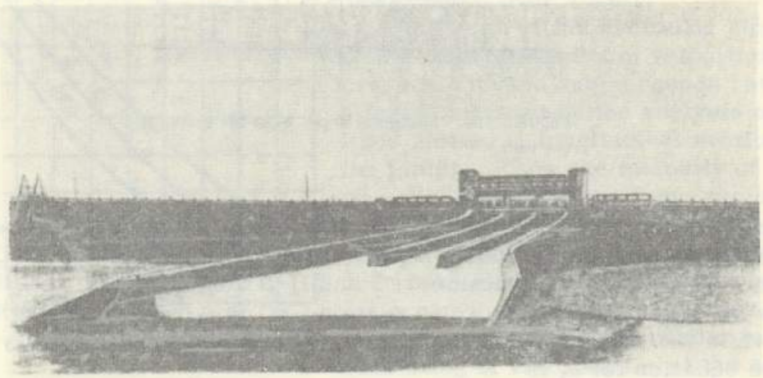


FIGURE 191. Overflow chute designed as a three-bay lock connected with the overfall sill and the tailrace by means of a special overfall shaft

As the investigations were limited by the conditions prescribed by the Management of the Levyi-Egorlyk power system for the discharge from the reservoir, they could not last longer than two hours at a time. It was therefore decided to pass several different discharges through one of the lock channels to plot the free surface curves at the sloping section, and to carry out visual observations of the streamflow near the chute inlet and at the sloping section. The investigations were carried out at the extreme right-hand channel of the lock. At several points along the channel staff gages graduated to 5 cm were affixed to the wall enabling the water depth to be read with an error of ± 2.5 cm. Each member of the investigating team measured the depth at one site. The measurements were made during the period of steady flow after each opening of the gate. The whole team then made visual observations of the streamflow (their results are not given here). Table 1 gives the results of depth measurements at the overall sill.

The discharge values were obtained from curves $Q=f(h_g)$ and $Q=f(H_0)$, obtained from the management of the hydro development. Let us compare the natural free surface curves with those predicted theoretically. At the break of the slope the (initial) depths were determined for the case of flow under a gate by means of Professor Charnomski's formula and, for the case of full gate opening, from formula

$$h_{cr} = \sqrt[3]{\frac{\alpha q^2}{g}}. \tag{1}$$

The gate was located 6 m from the upstream end of the sloping section of the chute.

TABLE 1

Site, No.	Distance between sites, m	Discharge Q, m^3/sec				
		15.0	28.0	40.0	59.0	70.0
		Opening of gate h_g, m				
		0.45	0.70	1.32	2.25	Full gate opening; head at entrance $H = 3.25$ m
		Depth at overfall, m				
1	14.5	0.30	0.45	0.77	1.15	1.75
2	14.5	0.57	0.63	0.90	1.22	1.25
3	14.5	0.45	0.55	0.80	1.03	1.30
4	14.5	0.32	0.40	0.58	0.85	1.12
5	25.0	0.25	0.33	0.65	0.77	0.94
6	29.0	0.28	0.40	0.62	0.80	0.97
7	44.0	0.30	0.40	0.57	0.80	0.95
8		0.30	0.42	0.55	0.82	0.90

The rated (theoretical) curves were calculated by the method developed by M. M. Skiba. The roughness coefficient for these calculations was taken as $n = 0.014$. Given the theoretical depth, we found the average flow velocity v. E. A. Zamarin's formula

$$h_{\text{air entr.}} = h\,(1 + 0{,}01) \tag{2}$$

was used for the stream depth with due allowance for air entrainment. With these data we can now plot the free-surface curves (Figure 192). Figure 192 shows the natural curves (solid lines) and the theoretical curves (dotted lines) of the free surface.

It was found that for the flow under a gate:

a) considerable differences between h_{th} and h_{nat} are observed only at the beginning of the sloping section. From the fourth site until the end of the sloping section, the difference, in most cases, does not attain 10%;

b) the results obtained by theoretical methods for calculating the depths in steep chutes, and particularly by Skiba's method, combined with E. A. Zamarin's formula for air entrainment in the chute, show good agreement with experimental data.

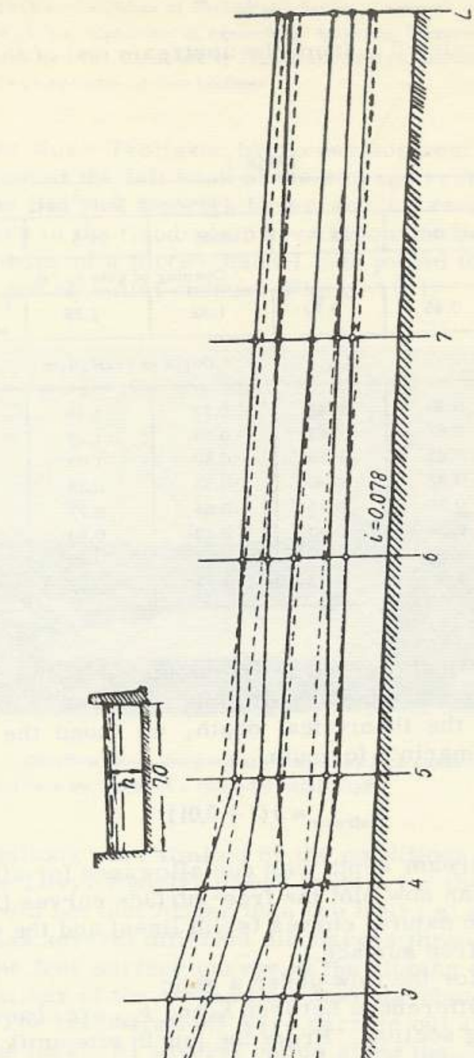


FIGURE 192. Free surface curves

NIMI CHAIR OF HYDRO STRUCTURES

Head: I. K. Fedichkin, Candidate of Technical Sciences, Lecturer

THE EFFECTIVENESS OF DIVIDING WALLS IN LARGE CHUTES

Responsible for Research: P. F. Kononenko, Candidate of Technical Sciences, Lecturer

Scientific Adviser: M. M. Skiba, Candidate of Technical Sciences, Lecturer

Research Team: V. P. Ivanov, Senior Lecturer
I. Kh. Ovcharenko, Junior Lecturer

Laboratory studies were carried out on the steep chute of the headworks of the Kuban'-Kalaus irrigation system. The chute is designed for a normal-water discharge of 740 m^3/sec and a flood-water discharge of 1240 m^3/sec.

These tests were conducted on a three-dimensional 1:60 scale model made of cement mortar; the tailwater channel was made of sand. The deformation of the sand channel served as an index of the effectiveness of the alternative designs, i. e. the actual material of the channel was not modeled. The tests included: 1) the study of the flow junction between the headwater and tailwater at different discharges and during operation of different channel sections; 2) the development of an effective design of the chute flume and its energy-dissipating devices.

In order to obtain a clear picture on the flow junction and the state of the tailwater, the tests were made with different combinations of operating outlets with completely opened bays. It was found that one of the most difficult operating conditions at the end of the chute corresponded to the flow in one of the outer sections. In this case the specific discharge was 20 m^3/sec and the flow velocity at the chute end reached up to 25 m/sec. The stream entrained a certain amount of water from the passive zone located in front of the nonoperating section. This led to the appearance of eddies far beyond the zone of slope protection.

The eddies in turn affected the active zone of the stream, pushing it toward the left or right bank (depending upon the degree of gate opening), thus causing a strong shooting flow. The specific discharge of the spilling (active) stream increased due to this pushing effect and to the entrainment of water from the passive zone. As a result, 3 to 6 min after the start of the model test, which corresponded to a 24-hour period under field conditions, considerable scouring of the channel bottom (up to 3 m under field conditions) could be noticed. Large amounts of sand from the channel bottom were entrained by the spilling stream and carried beyond the model limits. At the same time, the eddies formed in front of the nonoperating channel sections entrained sand particles, which were deposited on the apron, forming sand bars.

Analyzing the hydraulic pattern in the tailwater of the chute, the conclusion was drawn that the hydraulic regime in the tailwater, with asymmetrical operation of the outlet section, may be improved by reducing the specific discharge [i. e. discharge per length of spill front]. This proved feasible by removing the dividing walls in the chute. As a result, the streamflow joined the tailwater through a submerged hydraulic jump; the energy dissipation was increased as the jump spread over the whole width of the apron. Return currents and flash conditions, though noticed, were of markedly lower

intensity; scouring of the tailwater channel dropped to 0.7 m. Though return currents eventually deposited some sand on the apron, the amount was negligible. The amount of sand carried away beyond the model was also insignificant. When two, three, or four sections were in operation, the conditions at the tailwater area did not deteriorate and, on the contrary, showed a marked improvement due to the increased spilling length for normal as well as for flood discharges.

From these results it can be concluded that: a) the omission of dividing walls in the chute flume materially improved the conditions at the tailwater section which leads to the reduction of scouring in the tailwater channel; b) the omission of separating walls is advisable not only because it saves reinforced concrete (in our case, about 400 m^3) but it also improves the conditions at the flow junction.

The above recommendations were adopted by the Pyatigorsk Branch of the Yuzhgiprovodkhoz in the design of the final variant of the spillway chute.

LABORATORY INVESTIGATION OF THE ALEISK HYDRO DEVELOPMENT

Responsible for Research: I. K. Fedichkii, Candidate of Technical Sciences, Lecturer

Research Team: I. S. Khomenko, Candidate of Technical Sciences, Lecturer
S. K. Kuznetsov, Engineer
B. P. Avtonomov, Engineer

At the request of the Rostov Division of the State Design Institute "Vodokanalproekt" the Chair for Hydro Structures carried out laboratory research on two models of the Aleisk hydro development. One of the models built to a 1:60 scale represented the hydro structures; the other, three-dimensional, occupying an area of 548 m^2, simulated the hydro structures, a stretch of the Alei River, 4.5 km long, and part of its flood plain, 4.5 km long and 2.7 km wide. The horizontal scale of the three-dimensional model was 1:150, and the vertical scale 1:75.

The Aleisk hydro development includes:

1) a four-bay spillway dam (bay width 20 m);
2) aligned approach portion of river (width at bottom 60 m);
3) intake flushing basin;
4) water-intake structures and pumping station;
5) spillway earth dam (barrage), etc.

The works are intended for the passage of a flood discharge $Q = 1200\ m^3/sec$. The difference in elevation between the spillway sill and tailwater bottom is 0.30 m. The soil of the river banks and bottom consists of fine-grained powderlike sand.

The investigations were aimed at checking the correctness of the design solutions and the dimensions for the elements of the hydro development, as well as the correctness of the alignment of the approach stretch of the river.

The following problems were studied:

a) discharge capacity of the hydro structures;
b) regime of streamflow through the hydro structures;

c) possibility of scouring within the structures, and design and dimensions of scour-protection devices;

d) passage of ice, etc.

The investigations also covered the measurement of water stages, flow-velocity distribution, discharges, scour depth, direction of stream lines, etc.

In order to prevent formation of shooting streams in the headwater of the main river channel, and scouring of the channel slopes as a result of flood-flow contraction by flood-control dikes erected in the flood plain, and with the aim of improving the discharge capacity of the hydro structures, the alignment of the approach section of the river should be designed so that it coincides as far as possible with the direction of the stream through the flood plain, before it reaches the river-training structures. It is also necessary to build water-deflecting inlet dikes, and to ensure a more gentle slope of the channel banks upstream from the intake structures. The slope chosen in this case ranged between 1:3 and 1:10.

Tests for determining the discharge capacity of the structures showed that in spillway dams with small difference in level between the sill and the bottom of the tailwater channel, and for considerable flooding of the adjacent area, the discharge capacity does not increase as the result of increased spillway length, but as a result of an increase in the difference of level between the headwater and the tailwater. Thus, by increasing from three to four the number of operating outlet bays at the Aleisk development without changing the elevation of the headwater, the length of the spillway increased by 33%, while the discharge increased only by 6%. Therefore, where it is possible to increase the specific discharge of a spillway by a slight increase in the headwater level, it is advisable to reduce the length of the expensive spillway construction. At the Aleisk hydro development this was however impossible owing to the existence of fine-grained powderlike sand constituting the channel bottom.

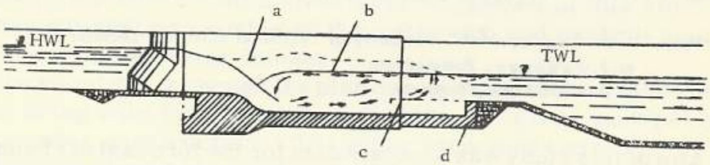


FIGURE 193. Flow junction between the headwater and tailwater at the stilling pool

a–flow junction between the headwater and tailwater sections with a stilling pool which does not extend beyond the dam piers (c); b–flow junction between the headwater and tailwater sections with a stilling pool, but extending a certain distance beyond the pier ends (d).

The fight against erosion downstream from dams such as the Aleisk dam, which has a small difference of level between the overfall sill and the tailrace bottom, is complicated by the fact that it is not always possible to adopt the traditional means for presenting shooting flow – various types of energy-dissipating devices. Thus, at the Aleisk hydro development, the installation of energy dissipators raises the headwater level, and this gives rise to malaria-breeding grounds during the dry season while, during the

flood period, adjacent settled areas may be inundated. Moreover, during ice drift, energy dissipators may cause ice jams to form at the dam. In rivers provided with levees, the increase in the headwater level could cause overtopping of the levees.

Good results in reducing erosion at such dams may be obtained by providing buckets (at the downstream apron]. At the Aleisk dam such a bucket reduced the amount of erosion by 33%. Moreover, the character of the scouring action became more gradual and less abrupt, which markedly reduced the danger to the structures.

The experiments showed that in dams with small differences of level between the spillway sill and the bottom of the tailwater channel, stilling pools must not extend beyond the dam piers. In Figure 193, a the stilling pool extends beyond the pier*, which, as was found, increased the turbulence of the spilling stream in the tailwater. On the other hand, by arranging the far edge of the stilling pool at a certain distance from the end of the piers (Figure 193, b), the level at the overflow sill drops and the flow junction is improved.

As a result of these investigations, the following changes were introduced in the designs: a new path for the realigned river stretch was established, the barrage and the island were omitted, a new design of guiding dikes was developed, the design of slope and bottom protection was changed both in the headwater and tailwater area, protection measures were revised, and other changes introduced.

NIIVT HYDRAULIC LABORATORY

LABORATORY INVESTIGATIONS ON THE TAILWATER SECTION OF THE NOVOSIBIRSK HEP

Responsible for Research: P. N. Orlov, Candidate of Technical Sciences, Lecturer

Research Team: F. M. Chernyshov, Candidate of Technical Sciences, Lecturer
M. P. Faddeev, Postgraduate
A. P. Kurbatov, Engineer

The aim of this study was to obtain data for the forecast of channel processes and ice-drift conditions in the tailwater of the Novosibirsk HEP, and to find ways of improving navigation conditions without impairing the operation of the municipal waterworks.

The investigation program provided for the study of the free surface configuration and of the stream kinematics for three discharge values: the calculation of ice-drift phenomena and channel changes; design of a system of flow-regulating structures; and tests on aerodynamic and hydraulic models.

A 35 km stretch of the river downstream from the HEP site was simulated at first on three aerodynamic models, and then (for the more complex structures) on two hydraulic models.

The aerodynamic models were made to two scales, $M_l = 1:2000$, and $M_h = 1:200$; while the two hydraulic models were made respectively to scales: $M_l = 1:600$ and $1:75$; $M_h = 1:500$ and $1:100$.

* [Obvious misprint, should read within the pier, see Figure 193, a.]

During the tests the following common measurement methods and techniques were used: level changes in the stilling basin were measured by means of needle gages, floating staff-gages and A. N. Losievskii's measuring devices; discharges were measured by means of triangular weirs; and streamflow was studied from photographs of the path of luminous floats.

The measurements on aerodynamic models were carried out by means of pneumometric tubes of the N. P. Gulyarov type, as well as by the simpler two-channel tubes.

Channel processes were investigated using A. V. Karaushev's, V. N. Goncharov's, and G. I. Shamov's relationships for the calculation of the river's load-carrying capacity by the method of balancing the silt deposits both over the length and the width of the river channel. These calculations took into account the soil composition in each stretch, the stream currents, and the changes in the soil composition during scouring.

The ice phenomena were calculated by two methods which made it possible to determine the movement of the ice edge as a function of the temperature, as well as the amount of frazil ice pushed below the ice cover as the edge of the ice moves downstream.

The forecast accuracy for channel processes and ice phenomena was confirmed by subsequent surveys and actual observations.

Model tests and theoretical considerations showed that, over the stretch from the HEP site to the sudden channel contraction in the Kameshek District, the scouring intensity is expected to vary, while further downstream, in the region of the Nizhne-Bugrinskii,shoals, silting may be expected to continue for several years. This will impair the nevigational conditions in the region of the Nizhne-Bugrinskii, Verkhne-Kudryashovskii shoals as well as at the Novosibirsk Creek shoals. The threatened erosion of the left river bank downstream from the inlet used by the Ob steamship line, as well as the erosion of the left bank river-training cut beyond the Otdykh Island may lead to interference with the normal operation of the existing water intakes and landing stages.

The system of river-training structures suggested in this study will improve the navigational properties of the waterway, and prevent sediment deposition in the existing water intakes.

It should be noted that although the plan has been approved by the customer, for the time being only the consolidation of the left river bank below the inlet mouth is under construction. The other structures are not yet being built, and this has already had unfavorable effects, such as channel changes and increased silt deposition at the right-bank water intakes.

Moreover, the Verkhne-Kudryashovskii shoals constitute s serious obstacle to navigation.

Investigations of the outlet of the downstream approach channel showed that during the initial period of operation of the Novosibirsk HEP the outlet section of the channel was situated in the region of increased sediment deposition.

The results obtained are in good agreement with the investigations carried out during 1952 by the VNIIG imeni B. E. Vedeneev. Of particular interest is the confirmation of results of tests conducted by the VNIIG laboratory on an inflatable aerodynamic model.

This study also gives an approximate estimate of the winter regime at the tailwater section of the Novosibirsk HEP, calculation of ice-free zones

on the stream, of ice and frazil discharge and of the zones of frazil-ice formation, and determination of the effect of daily regulation on frazil-ice formation.

LGMI CHAIR OF HYDRAULICS

Head: Professor B. V. Proskuryakov, Doctor of Technical Sciences

MORE PRECISE METHOD FOR CALCULATING THE ICE-COVER THICKNESS IN WATER PONDS AND THE HEAT TRANSFER THROUGH A SNOW-ICE COVER

Responsible for Research: V. A. Berg, Candidate of Technical Sciences, Lecturer

The present method of calculating the ice-cover thickness in water ponds is based on the assumption that the snow-ice cover provides a perfect insulation of the body of water from the atmosphere. According to this assumption, the increase of the cover thickness is determined by the difference between the thermal currents rising from the water depth toward the bottom edge of the cover and dissipating through it into the atmosphere. Therefore, any variation of this air temperature or, more exactly, of the thermal effect of the atmosphere on the snow-ice cover during the winter season, affects only the ice thickness without having any influence on the thermal conditions of the water body.

Temperature measurements in lakes show, however, that each cold wave in the air has a corresponding cold wave in the water body below the snow-ice cover which may be felt as far down as 10 m or more below the ice cover. This phenomenon is also connected with an accelerated growth of the ice-cover thickness, which is clearly visible while the ice cover is thin at the beginning of the winter, but also takes place throughout the winter season.

The cooling of the stagnant reservoir water proves that there is a transfer of heat from the water to the atmosphere through the ice cover. This may be explained by the fact that the rate of ice-cover growth, i. e. the rate of linear crystallization, being the rate of crystal formation of the ice lattice from the water molecules, is not infinitely great, but is subject to certain limits.

Laboratory tests carried out by numerous investigators: Tumlirtz, Volmer, Walton, Lilienthal, Hartman, Kost, et al.prove conclusively that the linear crystallization rate $v_{l.c.}$ depends on the temperature along the "crystallization front". Theoretical investigations by L. G. Kachurin, Volmer, et al. confirm these conclusions.

The temperature of the crystallization front of fresh water is always below zero. At zero temperature the water is in the state of phase equilibrium, i. e. there is no crystallization, but also no ice melting. The linear crystallization rate suddenly increases as the crystallization-front temperature drops.

Further analysis of this problem shows that the crystallization process in an open system, i. e. when the specific heat of crystallization is given the possibility of leaving the system, differs from the process in a closed, insulated system. Ice formation on inland water bodies occurs according to the open system.

Based on these laboratory researches and taking into account the extremely small degree of water supercooling, we assumed as a first approximation that the temperature of the crystallization front varies linearly with the heating (or cooling) load, according to the formula $t_{c.f.} \simeq 1/5000q$, where 1/5000 is a proportional and dimensional factor. The cooling load q of the snow-ice cover depends in turn on its thickness and on the temperature of the crystallization front and of the upper surface of snow or ice. According to the theory of heat transfer, the heating load $q_{theor} = K(t_{c.f.} - \theta_a)$, where θ_a is the actual air temperature equivalent to the total heat effect of the atmosphere and [solar] radiation on the snow-ice cover, and K is an over-all heat-transfer coefficient.

Numerous calculations and theoretical considerations confirmed the fact that the actual air temperature is usually higher than the thermometric value at the beginning and the end of winter owing to solar radiation, and lower in mid-winter owing to wind and radiant-heat dissipation from the surface.

According to traditional methods for predicting the ice-cover thickness, this depends on the heat load q_{theor} and is only limited by gradual decrease of q_{theor} as a result of the increase in the ice-cover thickness. In fact, the growth of the ice-cover thickness is limited by its linear rate of crystallization, for which we found the approximate expression:

$$v_{l.c.} \simeq -5.84 t_{c.f.} \text{ cm/hr} \simeq 0.00118q \text{ cm/hr},$$

where 5.84 is a proportional and dimensional factor.

Calculations showed that the linear rate of crystallization is always lower than the theoretical rate of ice-cover growth calculated from the value of heat release q_{theor}. Therefore, the calculation of probable ice-cover thickness with allowance for the rate of linear crystallization always results in values by 20-25% below the value for ice-cover thickness, calculated by traditional methods and comes closer to the actually observed values.

The difference in the heat fluxes

$$\Delta q = q_{theor} - q_{l.c.}$$

(where q_{theor} is the theoretical heat flow found from the above-mentioned formula; $q_{l.c.}$ is the heat flow corresponding to the linear rate of crystallization) will be dissipated through the snow-ice cover into the atmosphere, cooling the water mass of the reservoir. This excess heat flow sharply increases during cold waves and in turn creates cold waves in the water layers below the ice cover.

At present, Lengidep is carrying out calculations of ice-cover thickness according to the methods of this study, these data being necessary for the design of storage reservoirs.

KIIVKh CHAIR OF HYDRO STRUCTURE

Head: Professor V. V. Aristovskii, Doctor of Technical Sciences

SEEPAGE IN EARTH HYDRO STRUCTURES FOLLOWING A FALL IN THE HEADWATER LEVEL, AND ITS INFLUENCE ON THE STABILITY OF THE UPSTREAM FACES

Responsible for Research: Professor V. V. Aristovskii, Doctor of Technical Sciences

A number of earth hydro structures (dams, fish ponds, dikes, levees, canal and lock dikes, etc.) operate in conditions of intermittent and often rapid lowering of the headwater level, which causes seepage flows in the direction of the headwater, and affect the stability of the upstream faces. Soviet and foreign scientists (Shestakov, Charnyi, Reinius, et al.) proposed in recent years some methods based on the Boussinesq equation for calculating this type of seepage. From the analysis of the seepage-flow nets plotted by means of the electrohydrodynamic analog unit (EGDA), and from experiments carried out on the hydraulic integrator, the author arrived at the following conclusions:

1. The use of the Boussinesq equation for plotting the free-surface curves during the flow of water through the body of a hydro structure, cannot be justified.

2. In order to evaluate the stability of the upstream faces of homogeneous earth-fill structures made of sandy-clay soil, it is necessary to take into account the sudden lowering of the headwater level, whereby the body of the upstream slope is filled with water flowing by gravity under steady conditions (considering only the flow in the direction of the headwater).

3. In case of structures having an impervious diaphragm, only the seepage flow in the protective layer need be considered; this flow is characterized by outflow of water below the reduced headwater level, the rectilinear flow lines being parallel to the upstream face.

This study proposes formulas for checking the stability of upstream faces of earth-fill structures (without and with impervious diaphragms), which are exposed to the action of the seepage flow as a result of lowering the headwater level, the flow being directed toward the headwater.

For structures provided with impervious diaphragms this formula makes it possible: 1) to determine the permissible rate of lowering the headwater level without reducing the required safety factor for the stability of the upstream face; 2) subject to the same safety factor, to determine the permissible rate of lowering the free (flow) surface in the protective layer; 3) to determine the time interval required for lowering the headwater level at a certain definite rate but without impairing the stability of the upstream face.

The conclusions drawn in this study may be used in the design of those earth structures whose operating conditions require the reduction of the headwater level.

YuZhNIIGiM DIVISION OF HYDRAULIC ENGINEERING

Head: I. A. Chuprin, Candidate of Technical Sciences

THE AMOUNT OF SEEPAGE FROM TEMPORARY IRRIGATION SYSTEMS, AND ANTISEEPAGE MEASURES

Responsible for Research: I. A. Chuprin, Candidate of Technical Sciences

The irrigated fields of the Northern Caucasus foothills and particularly the fields of the Checheno-Ingush Autonomous S.S.R. have steep slopes reaching in some places values of 0.015 to 0.025. These fields have a thick pebble layer at a depth of from 0.6 to 1.5 m. The soil is very pervious, and the ground-water table is in most cases located at a great depth. Temporary irrigation systems in such fields lose large quantities of water through seepage.

This problem was first investigated at the Sunzha experimental reclamation station of the YuzhNIIGiM during the period 1950-1955.

The investigations were carried out on two soil varieties. The first, Ciscaucasian carbonate-containing chernozems developed on loess-like heavy loams with inclusions of small-size pebble, underbedded by large-size pebble to a depth of 80-150 cm. The second, Ciscaucasian weakly leached chernozem on loess-like heavy loams with inclusions of small-size pebble. The second variety is underbedded by pebble deposits of 65 to 80cm thickness. The seepage coefficient in the first soil variety is much smaller than that in the second.

Among the most urgent problems to be studied were: the determination of seepage losses from temporary irrigation ditches; reduction of water losses by proper compaction; determination of water losses for different flow velocities in temporary irrigation ditches and of the "useful seepage".

The seepage losses were calculated from measurements in Cipolletti (trapezoidal) weirs. The results are given in Table 1.

TABLE 1

Average values of seepage losses from temporary irrigation ditches for the period 1951-1958

Number of irrigation ditches	Soil variety	Average length of experimental plot, m	Average flow velocity of water, m/sec	Average water discharge, l/sec	Water losses in % per 100 m ditch length
4	I	550	0.70	50	2.17
2	I	300	0.46	35	2.81
2	I	300	0.35	30	3.70
2	I	250	0.20	30	4.85
4	II	550	0.72	50	3.15
4	II	300	0.65	40	3.86
3	II	300	0.52	40	4.10
3	II	200	0.20	30	7.40

The data in Table 1 confirm that in the second soil variety (other conditions being equal) the seepage losses are higher than in the first. But in both soil varieties the absolute losses are fairly high.

In investigating the efficiency of compaction, two parallel irrigation ditches, located 6 m apart, were tested simultaneously. One of the ditches was compacted at different soil-moisture contents, the other ditch was not treated in

any way and was used for the purpose of comparison. The degree of soil compaction was determined for a depth of 15 cm from the change in the bulk density. The ditch bottom was compacted by means of a special sliding steel compactor in the shape of an oblong metal box fitted to the cross section of the ditch (Figure 194). The box is filled with sand or dry soil. The side walls are hinged to the box bottom whose width equals the width of the bottom of the ditch.

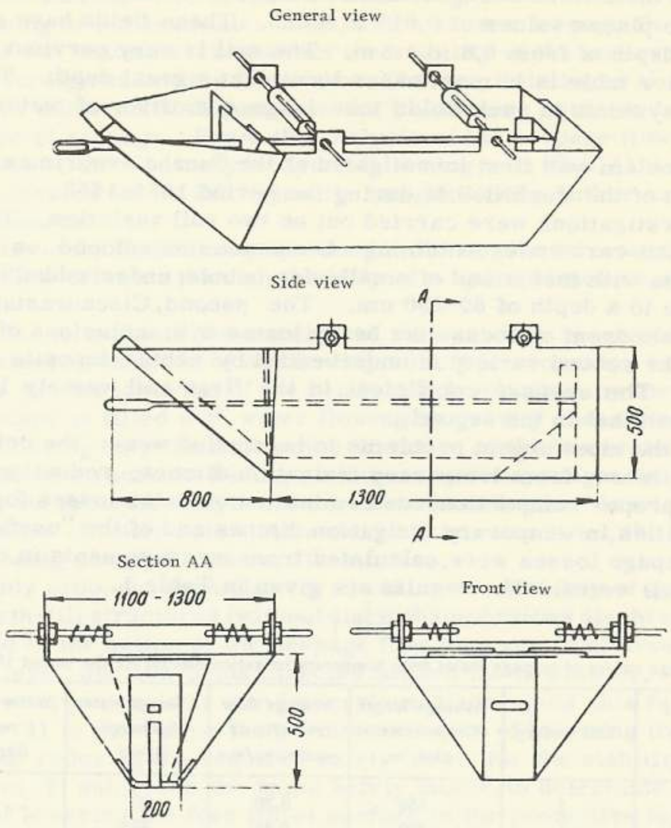


FIGURE 194. Metallic slide-type ditch-soil compactor

The compactor is attached to the tractor in line with the ditcher. The additional power required for traction of the ditch compactor is 10-12 hp. Table 2 shows the results of investigations for different discharges of water through the temporary irrigation ditches.

As can be seen from Table 2, compaction reduces the losses by a factor of 2 to 5. The degree of compaction depends on the soil moisture (Figure 195). Optimum moisture for hillside soil is 22 to 25% of a dry soil sample.

TABLE 2

Water losses from temporary irrigation ditches, determined by means of the soil compactor. for the period 1952-1955

Number of irrigation ditches	Characteristics of ditch channel	Average water flow-verlocity, m/sec	Duration of irrigation, hours	Discharge per ditch, l/sec	Losses per number of water discharges through the ditch			Factor of reduction of losses	Soil moisture in % after compaction
					1	2	3		
2	Compacted	0.70	26	40	0.4	0.43	-	7	25.2
2	Not compacted	0.67	26	40	2.8	2.3	-		
3	Compacted	0.56	24	35	1.5	1.6	1.8	2.5	20.0
3	Not compacted	0.52	24	35	3.72	3.70	3.6		
4	Compacted	0.39	25	32	1.18	1.21	-	2.9	23.0
4	Not compacted	0.36	25	32	3.40	3.31	-		
2	Compacted	0.21	24	30	2.91	3.17	-	3	28.1
2	Not compacted	0.19	24	30	8.8	8.6	-		

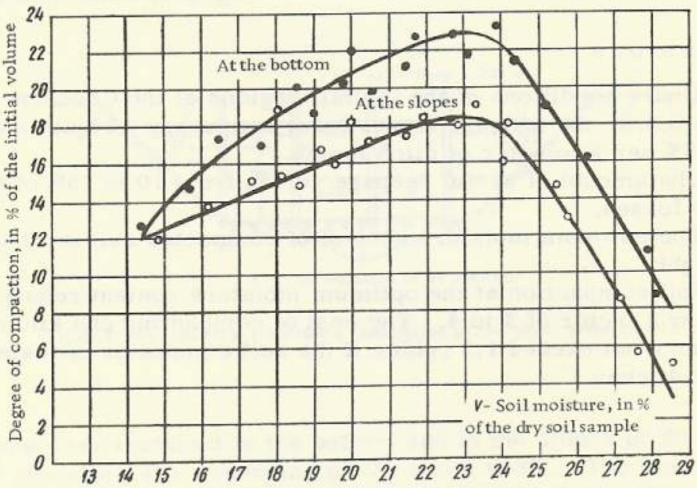


FIGURE 195. Curves of degree of soil compaction in temporary irrigation ditches, as a function of soil moisture

According to results of investigations carried out in the period between 1951 and 1955, the seepage losses in temporary irrigation ditches can be reduced by increasing the gradient of the ditch and hence the flow velocity to about 2-3 times the minimum figures. Test results are given in Table 3.

Part of the seepage water from the temporary ditches is actually not lost because it is used by those agricultural plants which are close to the ditches. This part of seepage water contributing to the increase in the moisture content of the soil layer below the ditches penetrated by roots is called "useful seepage". The amount of this useful seepage was found to be 10-15% of the over-all seepage water.

TABLE 3

Losses of water from temporary irrigation ditches in relation to the flow velocity

Number of ditches	Soil variety	Average parameters of the ditches			Seepage losses, in % per 100 m of ditch length	Increase in losses (compared with maximum flow velocity) by a factor of
		flow velocity, m/sec	slope	water discharge, l/sec		
1	I	1.06	0.0115	50	1.54	1.0
2	I	0.96	0.0104	40	2.27	1.47
3	I	0.71	0.005	40	2.80	1.82
4	I	0.59	0.0024	30	2.98	1.93
5	I	0.44	0.0017	30	3.52	2.28
6	I	0.22	0.0003	25	6.70	4.35
1	II	1.10	0.0117	30	2.42	1.00
2	II	0.99	0.0106	60	3.25	1.32
3	II	0.90	0.009	30	4.27	1.76
4	II	0.73	0.0062	30	4.72	1.95
5	II	0.55	0.0034	40	5.20	2.15
6	II	0.22	0.0008	20	6.21	2.48
7	II	0.12	0.0001	20	9.18	3.80

Conclusions

1. Under conditions of the foothill regions of the Chechen-Ingush Autonomous S.S.R. the seepage losses from temporary irrigation ditches amount to 25-45% per kilometer of ditch length.

2. The amount of useful seepage varies from 10 to 15% of the over-all seepage losses.

3. The optimum moisture content of compacted soil is 22 to 25% of the dry weight.

4. Soil compaction at the optimum moisture content reduces the seepage losses by a factor of 3 to 4. The cost of compacting one kilometer of ditch length does not exceed 1.5 rubles if the soil compactor operates together with the ditcher.

DETERMINING THE AMOUNT OF SEEPAGE LOSSES FROM THE AZOV MAIN CANAL

Research Team: I. A. Chuprin, Candidate of Technical Sciences
N. F. Savkina, Junior Research Worker

The Azov main canal in the Rostov-on-the-Don region has an over-all length of 92 km and a maximum discharge of 20 m^3/sec. The YuzhNIIGiM carried out between the years 1955 and 1958 investigations on seepage losses along the entire length of the canal.

These investigations had the following objectives:

1) determination of the hydraulic efficiency of the canal;
2) location of sites of increased seepage;
3) conclusions as to the necessity of antiseepage measures.

In 1955 the seepage losses were determined by means of Zh-3 current meters which measured flow velocity and discharge at different canal cross sections.

This method proved, however, to be unsuitable since the Azov canal, owing to its numerous water-retaining structures and inverted siphons operates over most of its length under backwater conditions. Moreover, the Zh-3 current meters are not very accurate, the measuring error reaching ± 5%, and it often happens that the measured discharge at the downstream site is larger than upstream.

Attempts to measure the discharge in the reaches between inverted siphons by the volumetric method without interfering with the operational conditions of the canal proved unsuccessful.

Another shortcoming of these methods is that they do not reveal the location of areas of increased seepage along the cross section of the canal.

The division for hydraulic engineering developed in 1956 a method for determining seepage losses by means of pipes, first described in 1931-1934 by P. D. Glebov and V. V. Vedernikov, but not further developed for practical use. This method can be used without interfering with the operation of the canal, and it allows the determination of seepage losses along both longitudinal and transverse sections of the canal bottom, and the exact location of areas of increased seepage.

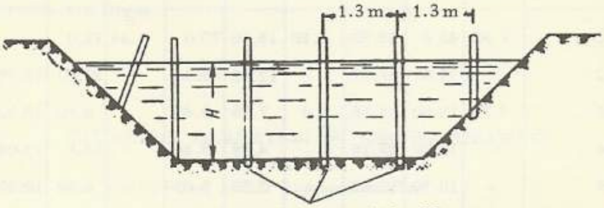


FIGURE 196. Diagram illustrating installation of the pipes in the canal

The pipes were installed in the bottom and in the slopes of the canal, the number of pipes per site depending on the canal width. The pipes were driven into the soil to a depth ranging from 6-8 to 25-30 cm (Figure 196). Thin-walled steel pipes were used having a diameter of 100 mm and a length of 1 to 3 m. In order to avoid compaction of the soil the lower end of the pipe wall was tapered.

The initial water level in the pipes was made equal to that in the canal.

The lowering of the water level in the pipes was observed by means of floats. Knowing the area within the tube walls on the bottom and on the slopes of the canal and the height of the column of water that seeped through during a certain time interval, it is possible to determine the volume of water in the pipes lost by seepage, and hence the seepage loss per unit area of the canal. Knowing the wetted perimeter at the measuring site, we can determine the seepage discharge per running meter of canal bed.

In calculating the seepage losses, allowance was made for a correction coefficient obtained from laboratory-field tests carried out by the hydraulic laboratory of the Institute in the flood plain of the Tuzlov River.

Seepage investigations by the above method were carried out in the Azov main canal between the years 1956 and 1958. During this period more than 100 transverse and longitudinal measuring sites were established over the whole length of the canal.

Flow lines for seepage below the canal bottom were plotted for each site. These graphs show the distribution of seepage losses over the canal cross section.

The sections of increased seepage, as found in 1958, coincide with the earlier data and are found on the following canal stretches: 80-165, 355-367, 433-452, and 703-832. In these stretches the seepage losses attain at times 40-80 g/liter/sec per km against the average canal values of 15-25 g/liter/sec per km.

Distribution of seepage losses in the bottom and slopes of the canal as measured in the stretches between the inverted siphons, for the years 1956-1958, are given in the table below.

TABLE

Seepage losses in the bottom and slopes of the Azov main canal in the sections between inverted siphons for the years 1956-1958

No. of section	Designation of canal section	Seepage losses in the section, liter/sec/km									Average seepage losses in the section, l/sec/km		
		right slope			bottom			left slope					
		1956	1957	1958	1956	1957	1958	1956	1957	1958	1956	1957	1958
1	PKO-D1	7.95	42.8	48.20	2.10	16.60	70.0	1.44	16.0	-	3.80	25.11	59.10
2	D1-D2	-	14.0	27.01	-	12.80	45.34	-	35.80	18.98	-	21.4	30.44
3	D2-D3	9.75	15.50	24.16	4.0	7.85	8.40	-	6.70	13.52	6.87	9.90	15.40
4	D3-D4	-	12.30	28.18	-	4.88	9.46	-	36.8	23.04	-	17.90	20.20
5	D4-D5	-	10.30	18.82	-	7.50	5.60	-	4.95	10.28	-	7.50	11.50
6	D5-D6	10.50	15.70	14.40	12.20	8.30	15.62	19.60	9.75	14.20	12.76	11.20	14.70
7	D6-D7	-	7.01	26.92	9.15	6.40	3.00	-	18.90	31.42	9.15	11.20	20.40
8	D7-D8	-	10.01	16.48	3.85	4.15	7.58	-	3.52	18.46	3.85	5.28	14.16
9	D8-PK920	-	7.10	-	-	21.80	-		16.00	-	-	15.00	-

The following conclusions may be drawn from this table:

1) the seepage losses in 1958 were higher than in previous years;

2) on the whole canal length the seepage losses in the right slope, which is made of fill material, are greater than in the left slope which consists of the natural soil. The seepage losses in the canal bottom are smaller than in both slopes except in the first and second sections.

3) the greatest losses were found in sections 1, 2, 4, 7.

The hydraulic efficiency of the Azov main canal was determined from data supplied by the management of the Azov irrigation system, relating to the duration of canal operation and the quantity of water delivered during one agricultural season.

The efficiency for the years under consideration, was as follows:

1956 – 0.848
1957 – 0.79
1958 – 0.68

The increase in seepage losses and the reduction of efficiency can be explained by the lowering of the ground-water table in this area as a result of the construction and operation (in 1958) of a distribution-discharge channel network.

Summary

1. The ground-water table in the area of the Azov main canal was lowered owing to the construction and operation of a distribution-discharge network in this area, which led to increased seepage losses and to a marked decrease in the canal efficiency.

2. The method developed by the hydraulic-engineering division, for determining seepage losses with the aid of pipes, was found to be fully satisfactory under field conditions. It permits, for any operational schedules of the canal, the calculation of over-all seepage losses, as well as the location of sites of increased seepage.

3. The water losses in sites of increased seepage, attaining 40-80 l/sec, i. e. 0.5 to 1% of the total discharge, though not yet disastrous, endanger the villages of Veselyi, Krasnyi Kut, Krasnoe Znamya and Us'man located below the bottom level of the Azov main canal.

The canal sections with excessive seepage have to be protected by means of special concrete linings.

AUTOMATIC REGULATION OF TUBULAR SPILLWAYS

Responsible for Research: N. O. Filippov, Senior Research Worker

In recent years precast structures have been widely used in irrigation systems of the Southeastern Regions of the RFSSR.

In the Volga-Don systems alone more than 6000 precast structures have been erected. These all-purpose structures are extremely practical and economical, and they lend themselves easily to mechanized methods of manufacture and erection.

Another advantage is their interchangeability, i. e. the possibility to obtain, by various combinations of 5 to 7 precast blocks, 5 to 6 types of hydro structures such as water outlets, crossings, baffles, spillways, inverted siphons, canal stormwater outlets, conduits under canals, etc.

YuzhNIIGiM and Yuzhgiprovodkhoz, carried out in the years 1956-1958 a series of investigations on the hydro structures of the Don system. It was found that variation of discharges through tubular spillways leads to changes in the hydraulic regime and hence to:

a) water-stage fluctuations in the headwater area with formation of dangerous backwater or recession conditions;

b) pressure fluctuations in the spillway conduit causing vibrations in the sections of the pipe and weakening of the joints;

c) unfavorable energy-dissipation conditions in the tailwater zone causing scouring, and interfering with the water-level regulation in the upstream intakes of lateral canals.

A radical way to improve the operation of tubular spillways is to ensure a hydraulic regime that would combine the positive features of stable headwater level and maximum discharge capacity. Such a mixed regime can be obtained by installing a simple automatic hydropneumatic [discharge] regulator (Figure 197). In such a mixed regime a jet of air is added to the water jet, thus creating a mixed air-water stream in the spillway tube.

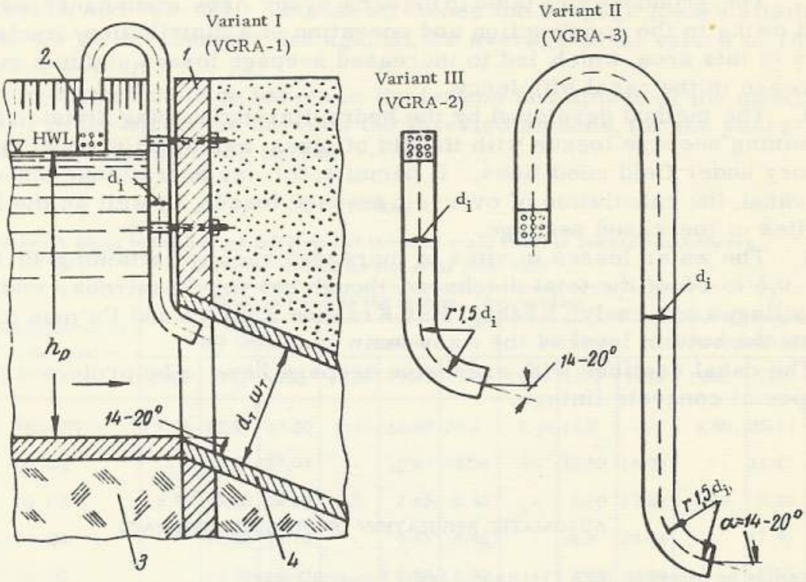


FIGURE 197. Schematic diagram of the VGRA-1 automatic discharge regulator

1-block "A"; 2-detachable perforated nozzle; 3-apron; 4-tube.

The regulator consists of a steel pipe with a diameter of $(0.19 - 0.2)d_T$ [which is the diameter of the spillway tube]. The upper end of the regulator, perforated or provided with a perforated nozzle, is placed at the desired stable headwater level, and the lower end is placed into the zone of maximum vacuum in the head of the structure.

The working principle of the automatic regulator is based on the control of pressure in the tube by feeding a controlled amount of air (depending on the water discharge) into the zone of maximum vacuum. Such a control system ensures the equalization of the discharge through the structure and in the canal, while keeping to a practically constant predetermined headwater level (Figure 198). With such an automatic regulator the mixed regime is brought about in the following manner.

During the starting period the discharge in the canal increases and the water level attains unhindered the inlet edge of the automatic regulator. During this period almost no vacuum is present in the canal head since the air flowing through the regulator tube breaks the vacuum. With the increase in the water level in the headwater area, the inlet to the regulator is submerged and the admission of air into the regulator tube is prevented. This

causes negative air pressure to appear at the canal head, increasing the pressure difference, thus increasing the discharge rate of the whole structure. With falling headwater level, the inlet opening of the regulator is again exposed, the air rushing in breaks the vacuum, reducing the pressure difference and, thus, the discharge rate.

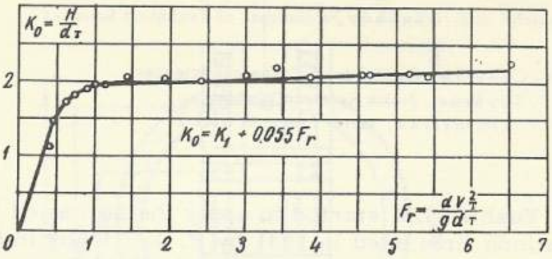


FIGURE 198. Degree of submersion of the tubular-spillway inlet provided with an automatic discharge regulator

The degree of inlet submersion K_0 may be found from the empirical formula

$$K_0 = \frac{H}{d_\text{т}} = K_1 + 0.055\,\text{Fr},$$

where K_1 = coefficient allowing for the position of the regulator inlet; we suggest to take for $K_1 = \frac{H_\text{NPL}}{d_\text{т}} = 1.6$ to 2.0;

$\text{Fr} = \frac{\alpha v_\text{т}^2}{g d_\text{т}}$ = Froude number of the pipe;

H = head at the spillway sill.

Field investigations of various designs of automatic discharge regulators, carried out over a period of two years at the Don irrigation system, permit us to affirm that the creation of a mixed hydraulic regime in tubular spillways with the aid of the described automatic regulators, markedly improves operations of these spillways. The automatic regulator ensures practically constant and predetermined headwater levels, even when the discharge varies by a factor of 2 to 3. It also ensures a constant hydraulic regime, and a steady flow through the upstream water outlets; it thus represents a first attempt to automatize irrigation operations.

HYDRAULIC LABORATORY OF THE YuzhNIIGiM

Head: A. D. Obukhov, Candidate of Technical Sciences

LABORATORY-FIELD INVESTIGATION OF SEEPAGE FROM PIPES

Responsible for Research: A. D. Obukhov, Candidate of Technical Sciences

Research Team: I. A. Chuprin, Candidate of Technical Sciences
N. F. Savkina, Junior Research Worker
Zh. V. Kruzhilina, Junior Research Worker

In 1956 the YuzhNIIGiM started to apply the so-called "pipe method" for field investigations first used in 1931 by P. D. Glebov in his investigations at the Golodnaya steppe, and later described by V. V. Vedernikov. This method gained wide acceptance in the years 1956 - 1958 for seepage and hydraulic efficiency investigations at the main canal of the Azov irrigation system. However, these investigations failed to clarify whether the seepage rate (v_p) determined by this method, depends on the diameter of the pipes d_p and the depth of penetration of the pipe into the soil (δ).

In his study, V. V. Vedernikov mentions only that "the pipe should penetrate into the soil to a sufficient but not excessive depth ... their diameter ... should be chosen as small as possible ..." and recommends this problem to be solved experimentally. V. V. Vedernikov evidently attached more importance to the water-level stability in the pipes than to the magnitude of d_p and δ.

P. D. Glebov's study also does not contain any more detailed indications.

Owing to these gaps in the "new" method the authors carried out in 1958 a series of laboratory-field investigations in order to establish:

a) the influence, if any, of the pipe diameter v_p and the penetration depth δ of the pipe on the seepage rate v_p;

b) the influence, if any, of the difference between the channel head H and the head in the pipe H_p, on the seepage rate v_p.

These investigations were conducted at the outdoor platform of the hydraulic laboratory in the flood plain of the Tuzlov River. The soils consist here of heavy loam and compact clays with a bulk density of 1.40 to 1.54, a porosity of 0.46 to 0.55 and a specific gravity of 2.61 to 2.97. The analysis of samples taken from a depth of 1.0, 1.5, and 2.0 m showed that the fraction < 0.001 predominates at all levels, the percentage of this fraction varying from 38 to 51. The fraction > 0.05 does not exceed 10 to 15%. The depth of the ground-water table was 1.60 to 1.70 m with a fluctuation of ± 10 cm.

Measuring pipes having a diameter d_p of 25, 50, 100, and 150 mm were placed in three cylindrical pits lined with reinforced concrete having an internal diameter of 1 m (see Figure 199). Four series of tests were made with δ = 2, 5, 10 and 15 cm. Each series consisted of several tests:

In test No. 2 we tried to establish the following relationship:

$$\eta = \frac{v_m}{v_p} = f(d_p;\ \delta), \qquad (1)$$

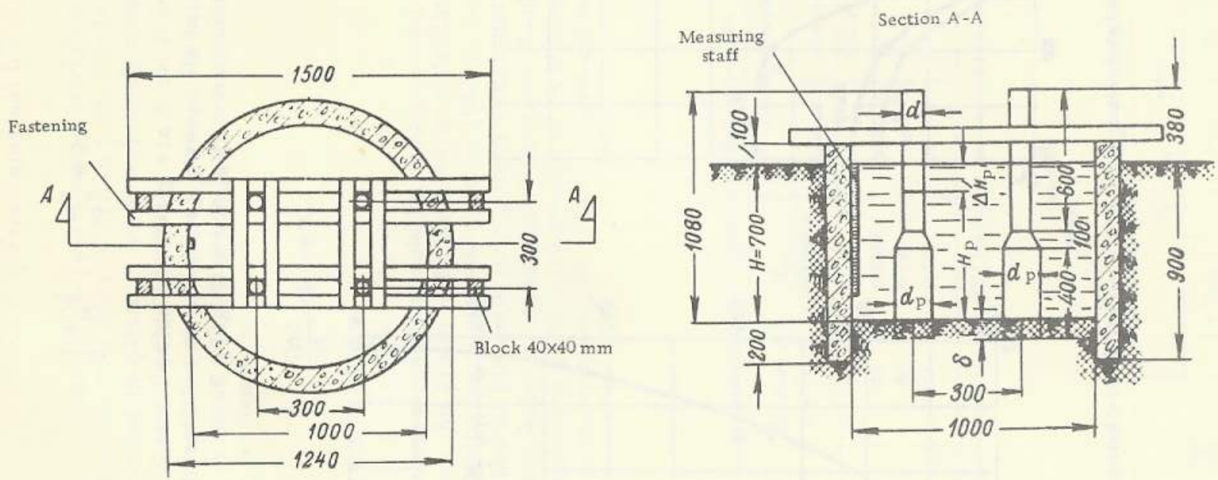


FIGURE 199. Location of pipes in the pits

where v_m = rate of seepage from the pit;
v_p = rate of seepage from the pipe.

Test No. 3 established the relationship

$$\eta_2 = \frac{v_m}{v_p} = \varphi\left(\frac{\Delta H_p}{H}\right). \qquad (2)$$

The results of test No. 2 are given in Tables 1 and 2.

As can be seen, in all cases v_p proved to be smaller than v_m, the coefficient η_1 being clearly connected with d_p and δ (Figure 200).

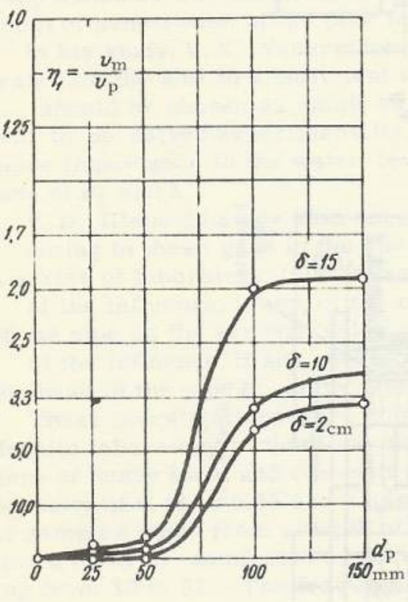


FIGURE 200. Relationship between d_p and η_1

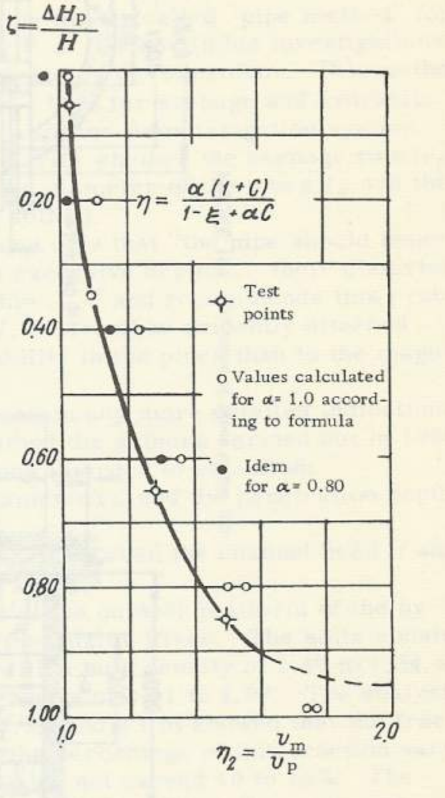


FIGURE 201. Relationship between $\frac{\Delta H_p}{H}$ and $\frac{v_m}{v_p}$

Figure 201 shows the relationship established by test No. 3 between $\frac{\Delta H_p}{H}$ and $\frac{v_m}{v_p}$, for pipes with d = 100 to 150 mm. The points from equation (3), obtained from Darcy's formula

$$\eta_2 = \frac{x_p (H + x_r)}{(H + x_p - \Delta H_p) x_r} = \frac{\alpha(1+c)}{1 - \zeta + \alpha c}, \tag{3}$$

where $H = \alpha x_p : x_r$; $\zeta = \Delta H_p : H$; $c = x_r : H$, agree fairly well with the test curve.

The following conclusions may be drawn from the investigations:

1. The pipe method is suitable for determining seepage losses from river channels and other bodies of water.

2. For river channels consisting of clays and heavy loams, the actual seepage rate at a relative drop of the water level in the pipes not exceeding 20% ($\zeta \leqslant 0.20$) can be found from

$$v_m = \eta_1 v_p, \tag{4}$$

where $\eta_1 = 3.0$ for $\delta = 10$ cm;
$\eta_2 = 2.0$ for $\delta = 15$ cm.

3. For $\zeta > 0.20$, equation (4) should be supplemented by the correction factor η_2, taken from the graph in Figure 201 or calculated from (3).

These relationships were already used in 1958 in field tests for determining the efficiency of the main [irrigation] canal and of a series of irrigation ditches of the Azov irrigation system in the flood plain of the Don River. The seepage pipes were instrumental in detecting sections of excessive seepage and in determining the places where antiseepage measures should be taken.

TABLE 1

Average rates of seepage from pipes and test pits

No. of test series	δ, cm	Duration of test, hours	V_m, m/24 hours	V_p in m/24 hours, with d_p			
				25 mm	50 mm	100 mm	150 mm
1	2	27	4.410	0	0.019	0.105	0.369
2	5	22	0.233*	0	0.098*	0.168*	0.250
3	10	50	0.740	0	0.003	0.037	0.040
4	15	351	0.104	0	0.003	0.043	0.044

REMARKS. Figures whose reliability is doubtful (as results from 4 series of test No. 2) are marked by asterisks.

TABLE 2

Experimental values of parameters (taken as average from 3 pits)

d_p, mm	$\eta_1 = V_m\ V_p$ with:		
	$\delta = 2$ cm	$\delta = 10$ cm	$\delta = 15$ cm
25	-	500.0	37.0
50	250.0	200.0	27.1
100	41.5	36.2	2.0
150	34.9	52.6*	2.0

Head: F. S. Salakhov, Candidate of Technical Sciences

PREVENTION OF SEEPAGE FROM THE BOTTOM OF STORAGE RESERVOIRS BY MEANS OF A LOAM APRON LAID BY THE HYDRAULIC-FILL METHOD

Scientific Guidance: F. S. Salakhov, Candidate of Technical Sciences

Research Team: B. M. Petrov, Engineer
A. M. Dzhashitov, Engineer

In connection with runoff-control works on a series of small rivers in the Azerbeijan S.S.R., it became necessary to develop effective measures for combating seepage from the bottom of storage reservoirs. In particular, while designing a storage reservoir at the Khachin-chai River, it was found that the flood plain to a depth of almost 70 m, consists of boulder and pebble beds where the seepage rate attains values of 50 m/24 hours. No storage of water in the volumes required by irrigation needs is possible without effective seepage prevention.

One method for preventing seepage is the placing of an hydraulic-fill apron on the bottom of the storage reservoir located within the highly permeable flood plain of the river. The apron was made of local loam available from a quarry at the left bank of the river.

The calculated length of the apron, taking into account the given conditions and the height (40 m) of the earth dam, was 700 m, for a width of 500 m. This result was checked on the electrohydrodynamic analog computer. In the design the thickness of the apron at the dam base was 2 m, and at the end of the apron 1 m. According to design data, the seepage coefficient at the apron should not exceed 0.0008 m/24 hours.

The task of our laboratory was to investigate the feasibility of forming an antiseepage apron by hydraulic fill with locally available material, and to establish the seepage coefficient and the necessary thickness of the apron.

To this end, the laboratory prepared and installed on a test area, located on the slope of a deep excavation pit, a special unit consisting of three seepage-measuring vessels, a pressure pipeline 315 m long, and a water reservoir installed at a height of 40 m above the seepage vessels. A special unit for preparation of slurry was installed in the area between the water reservoir and the seepage vessels, 5.2 m above the latter. Such an arrangement ensured gravity flow of slurry at a rate of about 1.5 m/sec. An additional, lower, reservoir was provided which reduced the pressure at the seepage vessels to about 7.5 m. The seepage vessels were 3 m high and had a diameter of 1.5 m. They were equipped with a sand-gravel filter, stopcocks for letting out the seepage water, and piezometers for measuring the pressure across the depth of the hydraulic-fill layer. Figure 202 gives a schematic view of this unit.

The seepage properties of the quarry material were tested on 88 samples. The soil proved to be of sufficient homogeneity.

The weighted-average size grade of the quarry materials was: 1.0 to 0.5 mm – 0.1%; 0.5 to 0.25 mm – 0.4%; 0.25 to 0.05 mm – 17.6%; 0.5 to 0.01 mm – 42.6%; 0.01 to 0.005 mm – 19.0%; 0.005 mm - 20.3%.

Filling of the field seepage vessels with soil was done under water (at a depth of 0.5 m and with a slurry concentration of 26% by weight. The slurry settled at a rate of about 7 cm/hr according to the condition of coagulation and settling; the seepage water was removed, and the whole process of filling repeated.

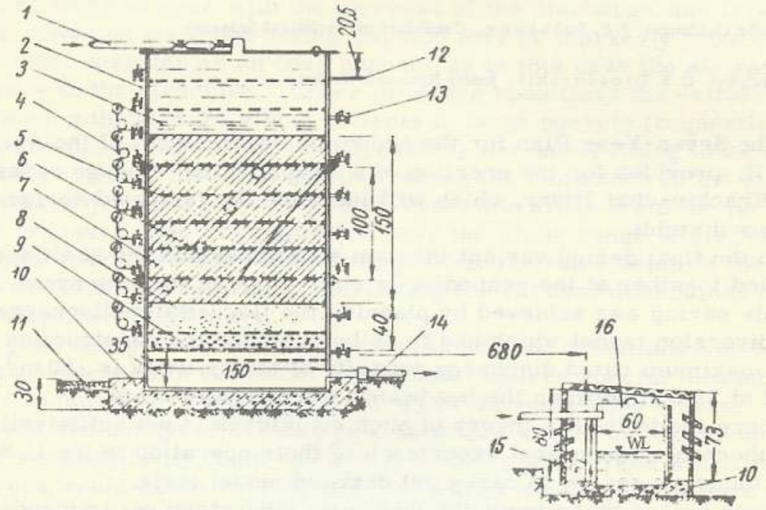


FIGURE 202. Field seepage-testing unit

1–pressure pipeline; 2–steel-sheet housing; 3–water; 4–pressure gages; 5–steel ribs; 6–pipes for piezometric gages; 7–hydraulically filled soil; 8–sand-gravel filter; 9–perforated bottom made of steel sheets; 10–concrete foundation; 11–gravel fill; 12–thermometer; 13–stopcock; 14–overflow pipe for seepage water; 15–wooden shed with shutters; 16–vessels for measuring the seepage water.

The first vessel was filled with ~ 2.8 ton of soil at a layer thickness of 1.1 m; the second, with ~ 3.7 ton, at a thickness of 1.5 m; the third, with ~4.3 ton, at a layer thickness of 1.9 m. The final thickness of the three soil layers was measured one month after filling, i.e. after self-compaction of the soil under its own weight and under a constantly kept water pressure, and was found to be 1, 0.6 and 0.2 m, respectively. The water pressure was then increased to 7.5 m, and 5 days later to 40.0 m.

The seepage flow was determined every 24 hours by the volumetric method

After the water head was brought to 40.0m, the seepage coefficient during the following week dropped sharply. In the following three months the seepage discharge varied very slowly from 0.00019 - 0.00021 to 0.00013 - 0.0015 m/24 hours.

The very favorable results of the tests and the suitable conditions for hydraulic filling permitted this method to be adopted in the working design of the storage reservoir. The design thickness of the apron was 1 m. The laboratory also carried out a study for the planning of the hydraulic-fill work. The detailed analysis showed the seepage-preventing facing apron, placed

by the hydraulic-fill method, to ensure savings of up to 30% and to speed up construction. The placing of this apron started in 1959.

EMERGENCY-SHAFT SPILLWAY AT THE STORAGE RESERVOIR ON THE KhACHIN-CHAI RIVER IN THE AZERBAIJAN SSR

Scientific Guidance: F. S. Salakhov, Candidate of Technical Sciences

Research by: O. S. Drozdovskii, Junior Research Worker

The Seven-Year Plan for the economic development of the Azerbaijan S. S. R. provides for the erection of a 20,000,000 m^3 storage reservoir on the Khachin-chai River, which will increase the irrigated surface of the Agdam district.

In the final design variant the dam structures and the shaft spillway are located together at the granodiorite right slope of the reservoir. A considerable saving was achieved by planning the flood-water discharge through the diversion tunnel which had to be built before the construction of the dam. The maximum rated discharge capacity of the spillway is 158 m^3/sec for a head of 39.3 m between the headwater and tailwater level.

Since the hydraulic theory of such structures is not sufficiently known, and there is no practical experience of their operation in the U. S. S. R., it was found necessary to carry out detailed model tests.

Even before these tests the hydraulic laboratory recommended the introduction of essential modifications in the shape of the entrance funnel, the vertical shaft and the elbow of the spillway. The design institute adopted these recommendations, and the design was then tested on a 1:46 scale model made largely of plexiglass.

The laboratory program included: 1) investigations and analysis of parameters and phenomena which do not lend themselves to analytical treatment; 2) working out measures for ensuring the optimum hydraulic regime at the various elements of the structures; 3) introducing improvements in the design of the structures.

To begin with, the laboratory investigated the problem of ensuring a smooth entrance of the water into the ring-shaped overflow, so as to avoid the formation of whirls and eddies, which would adversely affect the discharge capacity and the hydraulic regime of the spillway. The solution to this problem was complicated by the need to reduce the amount of excavation in rock, and this necessitated a departure from the conventional method of determining the contour of the reservoir bank and of the spillway intake.

The model tests showed that the proposed bank contour ensures proper flow conditions and uniform, specific discharges along the periphery of the ring-shaped overflow. The checking of the spillway capacity showed full agreement between the calculated and the actual discharge coefficients for a spillway operating under conditions of free (nonsubmerged) flow over the whole range of rated discharges.

The spillway outlet tunnel, its diameter having been designed for the discharge during the construction period, can operate both under pressure when the outlet is submerged, and without pressure for conditions of free outflow

from the tunnel. Further investigations were, therefore, conducted in order to study the effect of the streamflow regime in the outlet tunnel on the operation of the spillway as a whole.

Piezometric and vacuometric measurements were taken of the model in various sections of the inlet funnel, the vertical shaft, the elbow, and the beginning of the outlet tunnel. These measurements revealed the existence of a zone of pulsating negative pressures in the inner bend of the elbow. This zone tends to grow with the increase of the discharge, and it has a harmful effect on the elbow lining, but this may be markedly reduced if the outlet tunnel operates as an open channel, as in this case the air can circulate freely in the structure. Under pressure conditions the entrained air, after passing through the elbow, collects in large pockets frequently extending into the upper part of the tunnel, causing the tunnel lining to be subjected to variable internal stresses and increasing the vibrations in the structure.

Another objection against pressure operation arose from the fact that, to keep pressure conditions constant over the whole range of discharges, it would have been necessary to submerge a considerable depth of the outlet section of the tunnel by greatly increasing the vertical dimensions of the energy-dissipation chamber.

The final design provided for open-channel flow conditions which meant high flow velocities in the outlet tunnel and required special measures for efficient dissipation of the kinetic energy of the outflowing stream. On the other hand, the design of the energy dissipators should be chosen so as to exclude the possibility of the outlet tunnel being submerged which, for large discharges, would cause varying flow conditions and the appearance of a hydraulic jump.

At first the laboratory inclined toward the classical solution: energy dissipation by stilling pools and reduction of the outflow velocity to nonscouring values.

In view of the intermittent character and the short duration of shaft-spillway operation, the institute suggested a design in which the water stream flowing out from the tunnel would spread and be thrown clear of the rocky bank to alluvial deposits further downstream. This would prevent the scouring of the rock foundations of the structure.

The new design, the result of detailed laboratory research, consists of a chamber that widens in the horizontal plane and ends with an upturned bucket. With suitable contours of the bottom and the slopes, the inclined chamber ensures optimum flow conditions for the stream approaching the upturned bucket oriented with its convex side against the direction of flow. The shape of the upturned bucket ensures maximum spreading and throw-distance of the stream.

The data on model tests and laboratory design work were adopted in the working design of the structure now [1958] under construction.

VARIATIONS IN THE MINERALIZATION OF FREE-FLOWING GROUND WATER AT DIFFERENT DEPTHS

Responsible for Research: E. I. Zdobnov, Candidate of Technical Sciences

Investigation of free-flowing ground water in saline soils (the Mugan' steppe) both under natural conditions and in the vicinity of drainage schemes, showed the mineralization along the vertical to vary according to curves of different types.

Data on more than 45 boreholes varying in depth from 10 to 70 m showed that there are three characteristic mineralization curves for free-flowing ground water (Figures 203-205).

The curve of the first type of shows a maximum of mineralization at the free surface of the ground water, and a minimum at a certain depth h_d. This type corresponds to the zone of intensive salt accumulation in the soil layer above the ground-water table and can be observed where the ground-water flow is subject to backwater conditions, i.e. the seepage curve is, in fact, a backwater curve with a reduction in flow velocity along the stream.

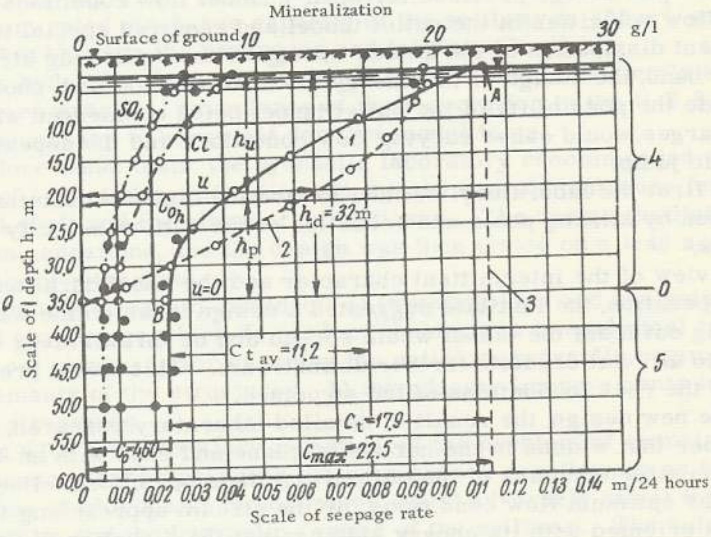


FIGURE 203. Mineralization curve of the first type

1 – solid residue; 2 – seepage-rate-parabola; 3 – datum plane; 4 – zone of flow; 5 – zone of stagnant water.

The second type differs from the first in that the maximum mineralization occurs at a certain depth h_d with a surface mineralization which is still fairly high, but slightly below the maximum. This type is also characteristic

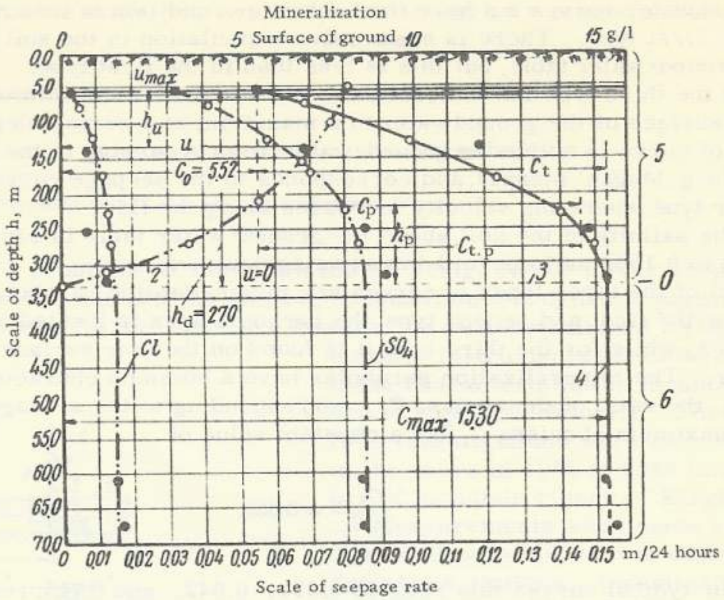


FIGURE 204. Mineralization curve of the second type

1–solid residue; 2–seepage rate-parabola; 3–datum plane; 4–zone of flow; 5–zone of stagnant water.

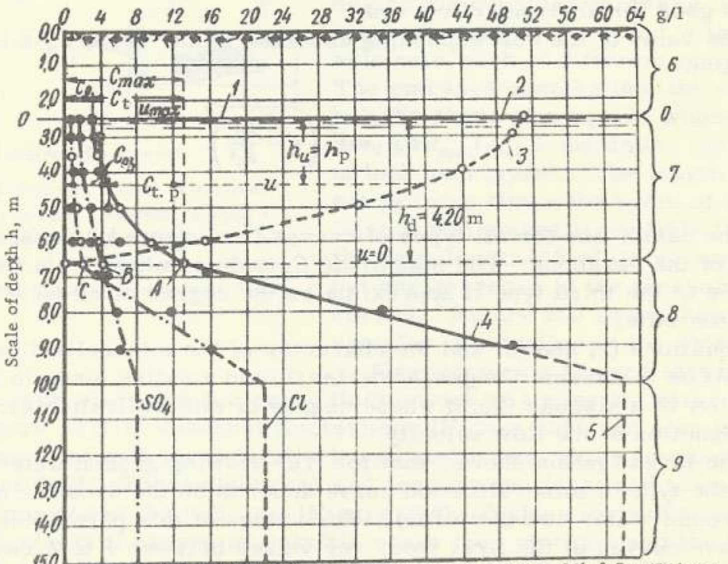


FIGURE 205. Mineralization curve of the third type

1–water level; 2–datum plane; 3–seepage-rate parabola; 4–tangent to the mineralization parabola; 5–solid residue; 6–zone in which flow starts with a rise of the ground-water level; 7–zone of flow; 8–transition zone where flow starts with the rise of ground-water level; 9–zone of stagnant water.

of backwater curves but here the backwater condition is less marked than in the first type. There is some salt accumulation in the soil layer above the ground-water table, but this is less than in the first type.

In the third type the mineralization increases from a minimum at the free surface of the ground water to a maximum at a certain depth h_d. This type of curve is noticed in ground-water flows occurring in the area of drainage (e. g. Mugan' steppe), and corresponds to the seepage curves of the backwater type where the velocity increases along the flow.

The salinity of the soil above the ground-water table is usually so low (0.10 to 0.15%) as to permit intensive agricultural use.

All of the three types of curves are mineralization parabolas.

For the first and second type, the parabola apex is located at a certain depth h_d while for the third type it is found on the free surface of the groundwater. The mineralization parabolas have a common characteristic feature: the ratio of the abcissa $C_{t.av}$ corresponding to the average depth h_d to the maximum abscissa C_t has a constant value of

$$\frac{C_{t.av}}{C_t} = 0.666. \tag{1}$$

For typical curves this ratio is 0.631, 0.642, and 0.713, respectively. The following equation:

$$C_{max} = C_t + C_0 \tag{2}$$

holds good for all types of curves.

The value of the corresponding abscissa to any depth h_p is found from equation

$$C_{t.p} = C_t\left(1 - \frac{h_p^2}{h_d^2}\right). \tag{3}$$

The datum line for all types of curves (i. e. depth h_d) passes through the apex of the parabola. The magnitude C_0 is determined from field data; for curves of the third type it approximates the degree of mineralization of irrigation water.

Equations (1) and (3) and the character of the mineralization parabola permit us to assume the ground-water flow in a saline medium to be a laminar flow of a viscous liquid whose degree of mineralization at a given depth is a function of the flow velocity.

The investigation showed that for free-flowing ground water in a saline soil the type of mineralization curve depends on the dynamic parameter of the ground-water flow ($2ki \times 100$). For values of this parameter below unity, we have curves of the first type; for values between 1 to 2, curves of the second type; and for values above 2, curves of the third type.

As a result of field investigations on free-flowing ground water in a saline soil, a reliable criterion (the so-called dynamic parameter $2ki \times 100$) was found, which should be taken into account in designing drainage systems for saline soils.

IEVKh OF THE ACADEMY OF SCIENCES OF THE KIRGHIZ S. S. R. LABORATORY OF HYDRO STRUCTURES

Head: K. F. Artamonov, Candidate of Technical Sciences

SOME DATA ON BED LOAD IN MOUNTAIN RIVERS

Responsible for Research: V. F. Talmaza, Junior Research Worker

In solving problems of water intakes for irrigation or power needs, we have to take into account the amount, composition and dynamic parameters of the bed load. So far, there are however no well-established methods or suitable instruments for determining these characteristics in mountain rivers of high flow velocity. Therefore, field investigations were carried out in the years of 1955 to 1958 in a number of mountain rivers of Kirghizia.

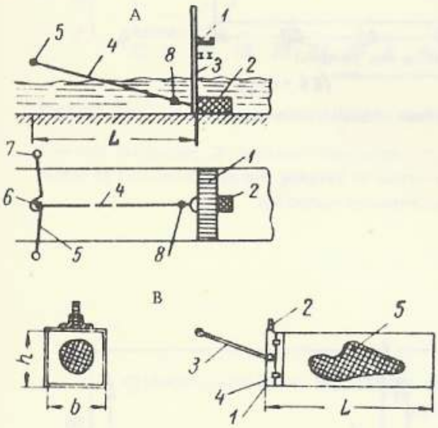


FIGURE 206. Diagram of a trap net

A. 1–bridge; 2–trap net; 3–rod; 4–tie-rod; 5–tie; 6–pulley or ring; 7–anchor; 8–ring; B. 1–mouth of trap net; 2–coupling for fastening the detachable rod; 3–hook for fastening the tie; 4–holes for mounting the detachable net; 5–detachable net.

Measurements were made at sites chosen, if possible, next to existing hydrometric stations. Investigations consisted in taking bed-load samples and determining the river slope, flow velocity, and discharge.

It was found that the most suitable device for bed-load sampling is the trap-net sampler provided with detachable nets, each of different mesh-size. The entrance opening into the sampler has the shape of a square whose sides are $1.5\,d_{\max}$ ($d_{\max}$= maximum diameter of bed-load grain). The length of the net is three times the width of the net inlet, which makes it possible to fill the sampler to 1/3 of its volume without any decrease in the measuring accuracy. Figure 206 shows how the sampler is installed in the stream.

Measurements showed that the bed load of mountain rivers is subject to fluctuations at a constant river discharge (Figure 207), to seasonal fluctuations (Figure 208), and also to fluctuations connected with the daily discharge variations, which frequently reach 100% (Figure 209). These discharge and load fluctuations lead to variations in the size grading of the bed load (Figure 209). Sudden torrential spring floods, particularly if caused by rainfall, cause mud streams and the increase of the bed load. Whereas the normal ratio of bed load to total load (suspended and bed load) in the mountain rivers of Kirghizia varies from 30 to 75%, it may reach 90% or more at the flood peak (Figure 208), and this distinguishes them from rivers in foothills and, even more, from lowland rivers where this ratio does not exceed 10%.

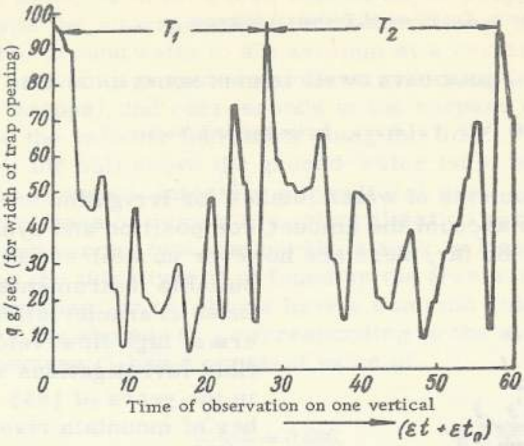


FIGURE 207. Fluctuation of the rate of bed-load transport in the vertical

T_1 and T_2 –fluctuation periods; t –time of keeping the sampler on the river bottom; t_n –time between samplings.

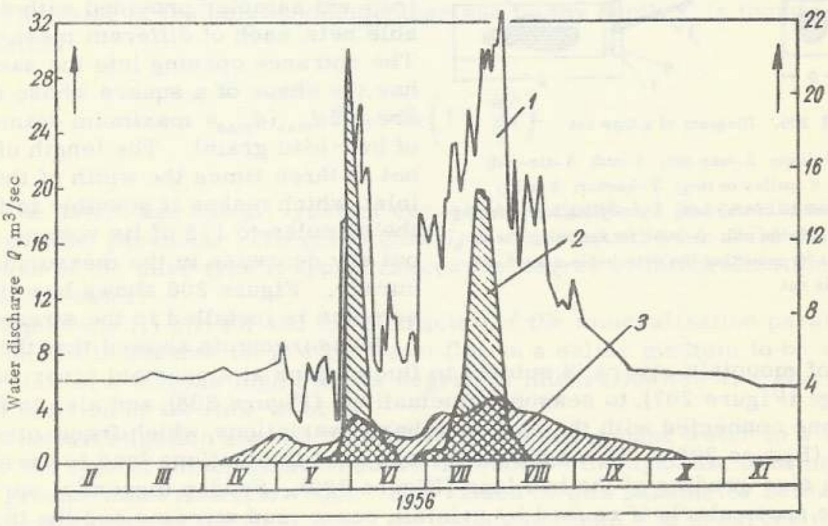


FIGURE 208. Seasonal fluctuations of water- and bed-load discharges

1–water-discharge hydrograph; 2–bed-load discharge hydrograph; 3–suspended-load discharge hydrograph.

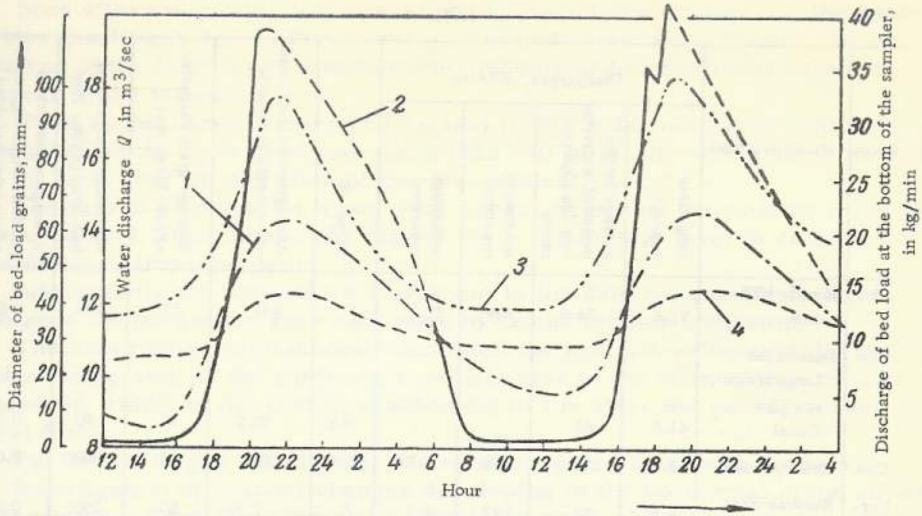


FIGURE 209. Daily fluctuation of water and bed-load discharges

1–water discharge; 2–bed-load discharge; 3–maximum diameter of bed-load grains; 4–average diameter of bed-load grains.

CHANNEL-FORMING PROCESSES IN LARGE WATER-INTAKE STRUCTURES ON KIRGHIZ RIVERS

The investigations were carried out on the largest water-intake works in Kirghizia (at the headworks of the Western and Eastern Large Chu Canals (LCC), and of the Chumysh, Buurda, and Kurshab hydro developments, which supply water both for irrigation and for power. The Chu and Kurshab rivers, where these hydro developments have been erected, flow through unstable channels and carry a considerable volume of large-size sediment loads. Great practical importance attaches therefore to problems connected with the channel-forming processes such as: silt deposition in the headwater area of the structure; extent of scour and of silt deposition in the tailwater section; the effect of channel changes on the design of the water intake; and the general character of the operation of the whole development and its constituent structures.

Table 1 gives brief hydrological characteristics of rivers in the area of developments which were investigated.

Channel deformations were observed by measuring twice a year the channel cross section in river stretches: in the headwater area over a length of 1.5 to 2 km; in the tailwater area over a length of 0.5 to 1 km.

The flow pattern in the horizontal plane was studied by means of floats; the composition of sediment load was checked by sieve analysis.

The investigations of channel-deformation processes in the headwater area revealed considerable silting upstream from low (3 to 4 m) dams.

TABLE 1

River	Hydro structure	Discharges, m^3/sec				Slope, %	Average annual bed-load movement, thousands of m^3	Average grain diameter of bed load, mm	Maximum diameter of bed-load grains, mm	Average water-intake coefficient during period of intensive bed-load movement
		long-time average	average for the period of intensive bed-load movement	maximum measured	minimum measured					
Chu	Buurda HEP dam	51.8	74.6	309	21.5	7	300	53	193	0.63
Chu	Dam of the Large Western Chu Canal . . .	41.9	62	-	-	3.8	30.5	30	90	0.53
Chu	Chumysh dam	29.9	60	159	6.72	1.8	20	40	300	0.60
Kurshab	Kurshab dam	26.8	57	197	6.9	7	7.36	50	250	0.34

TABLE 2

Large Western Chu Canal	Distance of measuring sites from the dam	m	25	85	145	205	267	332	487	800	932	1147
	Elevation of bottom above natural level	$\frac{Z}{\Delta H}$	0.964	0.968	0.946	0.914	0.835	0.816	0.774	0.591	0.5	0.368
Buurda HEP	Distance of measuring sites from the dam	m	25	55	49	237	289	394	429	524	739	911
	Elevation of bottom above natural level	$\frac{Z}{\Delta H}$	0.489	1.060	0.836	0.596	0.466	0.744	0.836	0.836	0.279	0.489
Kurshab dam	Distance of measuring sites from the dam	m	15	63	109	150	176	256	416	656	866	-
	Elevation of bottom above natural level	$\frac{Z}{\Delta H}$	0.846	0.824	0.811	0.824	0.840	1.00	0.769	0.730	0.550	-

ΔH – backwater caused by dam,

Z – absolute elevation of average bottom above natural level.

Soon after the putting into operation of channel-training works, the backwater conditions led to the appearance of new islands and shoals, and, at the end of the first flood, considerable amounts of bed sediment began to appear in the tailwater area.

Thus, at the Large Western Chu Canal (LWCC) the volume of sediment deposited in the first flood was more than half of all the sediment deposited in the first year of operation of the development.

The rate of silting after a two-year operation period can best be followed from Table 2, which shows the rise of the river-bottom level in relation to the distance from the dam.

Subsequently the channel area adjacent to the dam becomes stabilized, whereas the backwater zone continues to extend further upstream.

The expected erosion downstream from the hydro development did not materialize, and, on the contrary, a certain rise of the channel bottom was observed, which, at the LWCC, reached 0.8 to 1 m after the passage of the first flood.

Further silting of the channel bed was prevented by annual dredging.

Investigation of channel changes developing in the horizontal plane showed that average flood discharges in a regulated river channel cause formation of islands and shoals. In these cases the stream, as happened in the Large Western Chu Canal, tends to form bends whose direction is opposite to that provided in the design and this leads to the inflow of large amounts of bed load into the canal. The maintenance staff had therefore to carry out river-training works in the headwater area.

The putting into service of the Orto-Tokoi storage reservoir radically changed the conditions of winter operation of the Buurda HEP and the LWCC water intakes.

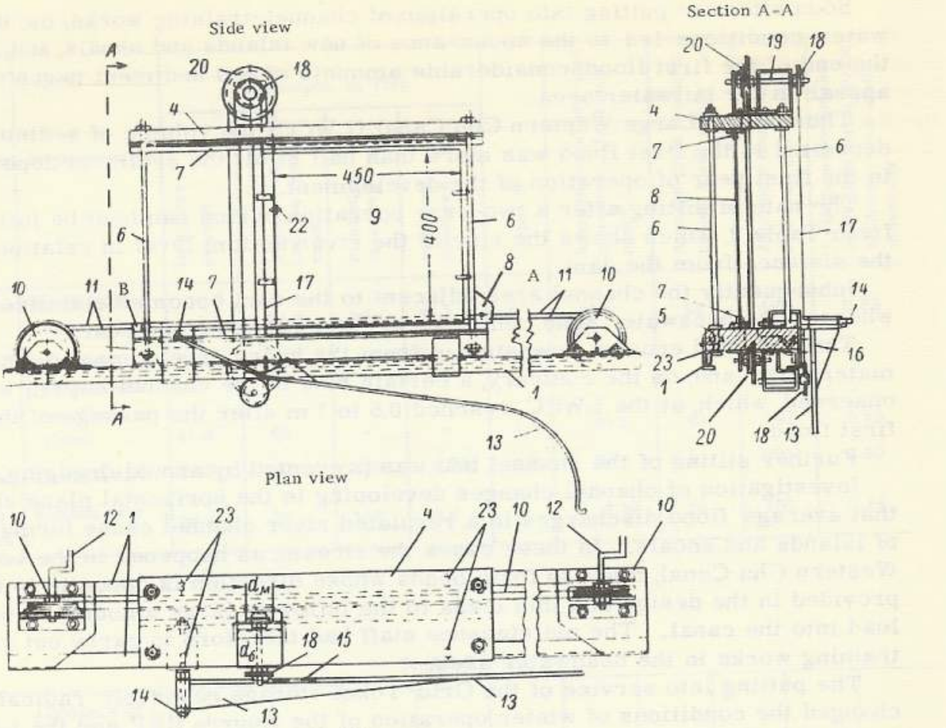
Observations at the LWCC showed reduction of sizes of slush ice to dimensions permitting the passage of the whole ice mass into the canal, where it melted after meeting the warmer source waters that were let into the canal.

At the Buurda HEP dam the size of the slush ice also diminished, but, at the same time, the winter water discharges dropped, which made it more difficult to move the slush ice into the tailwater.

In addition, there were short-lived, very intensive ice drifts as a result of the sudden inflow of frozen accumulations of slush ice and snow.

LABORATORY PROFILOGRAPH

Responsible for Research: K. F. Artamonov, Can



Scale 1:5

FIGURE 210. Profilograph

In the search for new methods of measuring deformations of model channels, the authors designed a highly accurate and efficient laboratory profilograph (a simplification of the profilograph designed by K. F. Artamonov in 1957 and used at the SANIIRI).

The new type of profilograph permits the automatic plotting of channel-model profiles directly on paper and to any desired, horizontal and vertical scale.

The profiles can be plotted without interrupting the flow of water through the model which is a particularly valuable feature for laboratory work. Other advantages of this device are: shorter time required for obtaining experimental data as compared with conventional methods, simple design, the possibility of manufacturing in small mechanical shops, low cost (1000 [old] rubles) and small weight (12 kg), and easy handling. There is no need for highly qualified staff, as the device can be operated by any average laboratory assistant.

The device (Figure 210) consists of a beam (1), made of two angle irons. The beam length is chosen according to the width of the model channel. The beam ends carry the two disks (10) for horizontal scaling, connected by ropes (11). The carriage (5) consisting of the upper (4) and lower (15) wooden crosspieces connected by bolts (6), moves on roller bearings (23) along the beam. Frame (8) together with the detachable board (9) moves in the guides (7) made of angle irons. At B the carriage (5) is fastened to cable (11) of the small pulley (10), while the board (9) is fastened at A to the

cable (11) of the large pulley (10). Carriage (5) is also equipped with a rod (13) and a pulley block system for vertical scaling (pulley blocks (18), (20)) with cables which are fastened to the recording device (22).

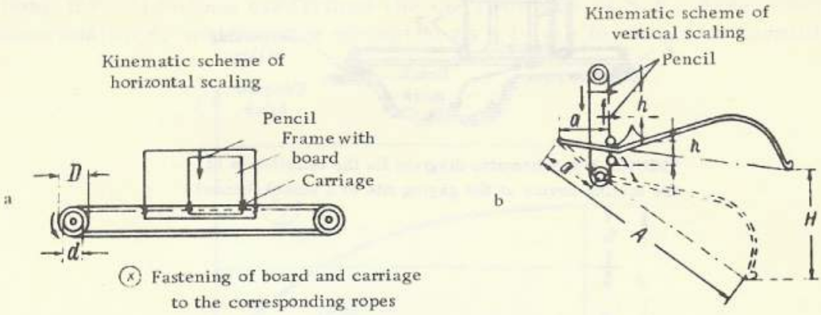


FIGURE 211. Kinematic scheme of horizontal (a) and vertical (b) scaling

The principle of horizontal and vertical scaling is based on the relationships

$$M_{\text{hor}} = \frac{D}{d} - 1,$$

$$M_{\text{vert}} = \frac{a}{A}$$

and is easily understood from the kinematic schemes in Figure 211.

The exact setting of the vertical scale can be obtained by the element (15) shaped according to a profile curve with $\rho = 1 - \cos\alpha$ (in polar coordinates); the roller (16) fastened to cable (17) moves over this curved element.

The horizontal scale may be altered by changing the pulleys (10), and the vertical scale, by shifting the cable from the large pulley (20) (diameter d_1) to the smaller pulley (d_s) (see Figure 210).

Vertical scaling follows the relationship

$$M_{\text{vert}} = \frac{d_1}{d_s} \frac{a}{A}.$$

The plotting of profiles (Figure 212) consists in mounting the beam (1) together with carriage (3) on the river-bank pillows (2), installing the rod (13) on the fixed gaging staff of the profile to be plotted, and, finally, in shifting the scaling device along the beam (1), by means of wheel (12).

The whole operation from the moment of placing the device on the model channel until the plotted profile is obtained takes only 2 to 4 minutes.

The simplicity of the device and the ease with which the profiles are plotted, permits the investigator to obtain a clear picture on channel transformations during the tests without interrupting the flow through the channel,

and also to study the optimum operating conditions of hydro structures in a channel subject to morphological changes.

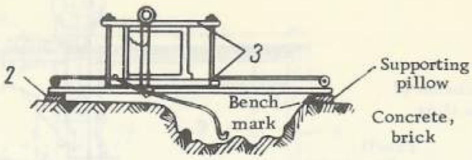


FIGURE 212. Schematic diagram for the installation of the scaling device at the gaging site of a model channel

ENIIZiM DIVISION OF HYDROLOGY AND RECLAMATION

Head: G. V. Karaus

RESEARCH ON CALCULATION OF DRAINAGE SYSTEMS IN NONHOMOGENEOUS SOILS

Responsible for Research: V. A. Ionat, Candidate of Technical Sciences, Senior Research Worker

Research Team: A. G. Toom, Junior Research Worker
U. Kh. Tomberg, Junior Research Worker

The large-scale development of reclamation works and the transition from open-ditch to underground drainage require more exact calculation methods. The existing methods which assume a homogeneous soil may lead to completely erroneous results.

This refers particularly to drainage systems in stratified soils, such as double-strata deposits, shallow peat bogs underbedded by mineralized soils whose seepage coefficients differ sharply from that of peat, etc.

In order to perfect the existing seepage-calculation methods and to develop formulas allowing for soil inhomogeneity, the ENIIZiM carried out laboratory and field investigations. These included: verification of the formulas derived by Kh. A. Pisar'kov and D. P. Yunevich, which were found to be based on erroneous theoretical assumptions, and not suitable for practical application; experimental laboratory check (on the electrohydrodynamic analog unit) of V. S. Kozlov's recommendations for the calculation of "imperfect" drains; and particularly the following expressions for discharges from zones below the drains, which were found to be incorrect:

$$q_0 = \frac{K}{PS} BH^2, \tag{1}$$

$$B = 1 + 5{,}5 \sqrt{\frac{(H_0 - a)\, r}{H_0 a}},$$

where H_0 = thickness of the aquifer;
a = height of drain above aquifer bottom;
r = radius of drain;
P = seepage modulus;

These investigations confirmed the correctness of S. F. Aver'yanov's recommendations, which were accepted as a basis for the new formulas.

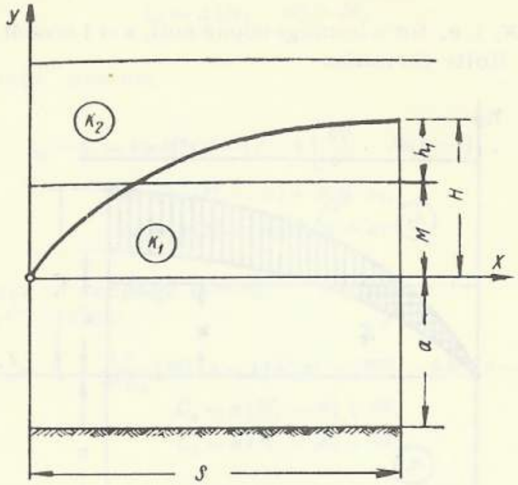


FIGURE 213. Graph for determining the arrangement of drains in the lower [soil] stratum for steady-flow conditions

Proceeding from Dupuis' equation, N. K. Girinskii's functions and S. F. Aver'yanov's conclusions, the investigators derived formulas for calculating the discharges through "perfect" and "imperfect" drains located in nonhomogeneous soils with seepage and ground-water inflow under steady-state conditions.

The discharge formulas obtained for such conditions were used in deriving the differential equations for the water balance, used in the method of successive changes of the steady state. It was thus possible to calculate drains operating under unsteady conditions.

For cases of evaporation or seepage the new formulas permit solutions to be obtained by the trial-and-error method. A special method was devised to speed up the computation work.

In view of the practical importance of these formulas for two-layer soils, they are given below.

When drains are laid in the lower stratum, having a seepage coefficient K_1 (see Figure 213) the distance between the drains, for steady flow, is found from the following formula:

$$E = 2S = 2\sqrt{\frac{K_1}{P}\left\{nH^2 + 2\left[a\varphi - M(n-1)\right]H + M^2(n-1)\right\}}, \qquad (2)$$

where $n = \frac{K_2}{K_1}$; $\varphi = \frac{1}{1 + 2{,}94\frac{a}{S}\log\frac{2a}{\pi d}}$;

P = rated value of seepage modulus;

d = drain diameter in m.

For "perfect" drains with $a = 0$, we obtain

$$E = 2\sqrt{\frac{K_1}{P}\left[nH^2 + (n-1)M(M-2H)\right]}. \tag{3}$$

For $K_2 = K_1 = K$, i.e. for a homogeneous soil, $n = 1$ and $M = 0$. Formula (3) is related to the Rotte formula.

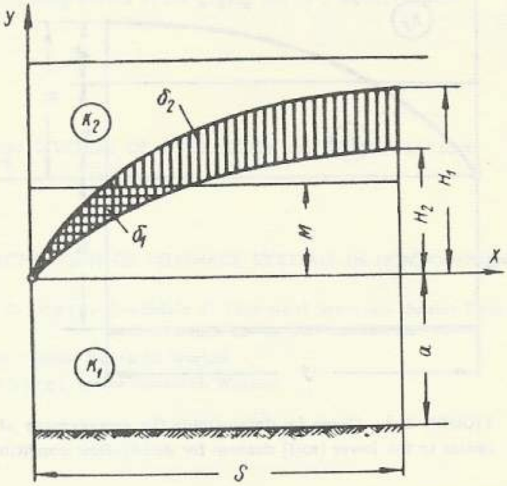


FIGURE 214. Graph for determining the arrangement of drains in the lower stratum for unsteady-flow conditions

For unsteady-flow conditions (Figure 214) the distance between the drains is found from

$$E = 1{,}60\,\alpha\sqrt{\frac{K_1 T\sqrt{\Delta_n}}{\delta_c A}} \quad \text{for } \Delta_n > 0, \tag{4}$$

$$E = 2{,}25\,\alpha\sqrt{\frac{K_1 T\sqrt{-\Delta_n}}{\delta_c B}} \quad \text{for } \Delta_n < 0, \tag{5}$$

where $A = \arctan\dfrac{2n(H_1 - H_2)\sqrt{\Delta_n}}{\Delta_n + 4C_1C_2}$;

T = duration of operation period;

$B = \ln\dfrac{(2C_1 - \sqrt{-\Delta_n})(2C_2 + \sqrt{-\Delta_n})}{(2C_1 + \sqrt{-\Delta_n})(2C_2 - \sqrt{-\Delta_n})}$.

The magnitudes Δ_n, C_1 and C_2 are determined according to the type of flow supply and the arrangement of the drain.

1. In the absence of seepage and evaporation, we obtain:

a) for "perfect" drains:

$$\Delta_n = \Delta_1 = 4\,M^2(n-1),$$
$$C_1 = n(H_1 - m) + M,$$
$$C_2 = n(H_2 - m) + M;$$

b) for "imperfect" drains:

$$\Delta_n = \Delta_2 = 4\,nM^2(n-1) - 4\left[\frac{a\varphi}{\alpha^2} - M(n-1)\right]^2,$$
$$C_1 = n(H_1 - m) + m + a\varphi,$$
$$C_2 = n(H_2 - m) + m + a\varphi.$$

2. For the case of seepage ($\varepsilon = P$)

a) for "perfect" drains:

$$\Delta_n = \Delta_3 = \frac{4\,n}{\alpha^2 K_1}\left[M^2(n-1)\,K_1\alpha^2 - PS^2\right] - 4M^2(n-1)^2,$$
$$C_1 = n(H_1 - m) + M,$$
$$C_2 = n(H_2 - m) + M;$$

b) for "imperfect" drains:

$$\Delta_n = \Delta_4 = \frac{4n}{\alpha^2 K_1}\left[M^2(n-1)\,K_1\alpha^2 - PS^2\right] - 4\left[\frac{a\varphi}{\alpha^2} - M(n-1)\right]^2,$$
$$C_1 = n(H_1 - m) + M + a\varphi,$$
$$C_2 = n(H_2 - m) + M + a\varphi.$$

3. For the case of evaporation ($\varepsilon = e$)

a) for "perfect" drains:

$$\Delta_n = \Delta_5 = \frac{4n}{\alpha^2 K_1}\left[M^2(n-1)\,K_1\alpha^2 + eS^2\right] - 4M^2(n-1)^2,$$
$$C_1 = n(H_1 - m) + M,$$
$$C_2 = n(H_2 - m) + M;$$

b) for "imperfect" drains:

$$\Delta_n = \Delta_6 = \frac{4\,n}{\alpha^2 K_1}\left[M^2(n-1)\,K_1\alpha^2 + eS^2\right] + 4\left[\frac{a\varphi}{\alpha^2} - M(n-1)\right]^2,$$
$$C_1 = n(H_1 - m) + M + a\varphi; \qquad C_2 = n(H_2 - m) + M + a\varphi.$$

The weighted mean value of water yield δ_c is found from

$$\delta_c = \delta_1\omega + \delta_2(1-\omega),$$

$$\omega = \frac{M(H_1 - \sqrt{H_1^2 - M^2})}{H_1 H_2},$$

δ_1 = the water yield of the lower layer;

δ_2 = the water yield of the upper layer;

$$\alpha = \sqrt{\frac{[M^2 + 2M(H_c - M) + n(H_c - M)^2](M + H_c)}{H_c^2[2M + n(H_c - M)]}},$$

$$H_c = \frac{H_1 + H_2}{2}.$$

Taking $n = 1$ and $M = 0$, we obtain formulas for homogeneous soils.

The studies described in this paper contribute to the improvement of drainage calculations for nonhomogeneous soils. The results were prepared for publication and handed over to the design institutes for further use.

GIDRORYBPROEKT OF THE STATE PLANNING COMMITTEE OF THE RSFSR

THE USE OF NAVIGATION LOCKS FOR THE PASSAGE OF FISH FROM THE TAILWATER SECTION OF THE HYDRO DEVELOPMENT INTO THE STORAGE RESERVOIR

Responsible for Research: Z. M. Kipper, Senior Hydraulic Engineer

Research Team: Yu. A. Mitrokhina, Senior Specialist for Fish Breeding
I. A. Dergacheva
M. Ya. Stepen'kova, Senior Engineer
S. F. Kovaleva, Fish-breeding Specialist
M. N. Kotsarenko, Fish-breeding Specialist
S. F. Kovaleva, Fish-breeding Specialist
Z. I. Zhukova, Fish-breeding Specialist
V. N. Petropavlovskaya, Fish-breeding Specialist
V. I. Shvedikova, Fish-breeding Specialist
N. I. Golova, Senior Technician-Hydrologist

With the development of hydro construction on U. S. S. R. rivers, the problem arose of finding ways and means of compensating for the damage caused by these structures to the fish reserves in inland water bodies. One of the principal means used to this end is the construction of fish passages from the tailwater area of the dam to its headwater area, i. e. to the spawning grounds.

In existing hydro developments when no fish passages are provided, navigation locks can be used for this purpose.

In the years of 1957 and 1958, the Gidrorybproekt conducted a number of investigations and experimental studies on the passage of fish to the spawning grounds through the following navigation locks: Tsimlyanskii (Don River); Ust'-Manych (Manych River); Aparin (Severnyi-Donets River); and Kegums (Daugava River).

The studies included:

a) laboratory and theoretical research on models of the Tsimlyanskii and Stalingrad locks;

b) field observations and experiments at the Tsimlyanskii, Ust'-Manych Aparinsk and Kegums locks.

One of the basic factors affecting fish migration is the flow velocity and the water temperature of a river. Fish always move against the current and they strive to reach warmer water.

The tests at the navigation locks involved the study of various flow velocities both in the approach navigation channel and in the lock chamber. The main task was to ensure such a velocity in the downstream approach channel of the lock that fish would be attracted from the river into the channel.

The flow velocities obtained in these studies are given in Table 1.

TABLE 1

Name of navigation lock and rate of flow of water during the emptying of the lock chamber	Flow velocity at the outlet from the approach channel into the river, m/sec	Flow velocity in the river, m/sec
1. Tsimlyanskii (on the Don River)		
a) at a flow rate of 65 m^3/sec	0.75	0.76
b) at a flow rate of 36 m^3/sec	0.40	0.76
2. Ust'-Manych (on the Manych River)		
a) at a flow rate of 10 m^3/sec	0.20	0.14-0.20
b) at a flow rate of 1 m^3/sec	0.10	0.14-0.20
3. Kegums (on the Daugava River)		
a) at a flow rate of 25 m^3/sec	1.20	1.20
b) at a flow rate of 10 m^3/sec	0.56	1.20

Figure 215 shows the flow pattern at the Kegums lock on the Daugava River.

From these data it can be seen that there is always a possibility to create in the approach channel such flow velocities as will attract the fish from the river into the channel (Figures 216 and 217).

Fish attracted into the approach channel were found to move through the lock chamber even at channel flow velocities of 0.1 to 0.3 m/sec.

From the approach channel the fish are attracted into the lock chamber by the water passage from the upper lock chamber or the upstream storage reservoir (with open downstream lock gates), the flow velocities usually not exceeding 0.3 m/sec in the upper chamber and 0.1 m/sec in the lower chamber (Figure 218)

After emptying the lock chamber the downstream gates remain open. The water-conveying galleries in the upstream end (head) of the locks start to operate and the water passes through the whole chamber. This water discharge increases the flow velocity both in the lock chamber and at the downstream approach channel intake (Table 2 refers to the Tsimlyanskii lock).

TABLE 2

Lift height of gate	Flow velocity in the lock chamber (m/sec)	
	at the upstream end	at the entrance into the downstream channel
0.75 m – large discharge	0.70	0.45
0.40 m – medium discharge	0.40	0.25
0.25 m – low discharge	0.20	0.10

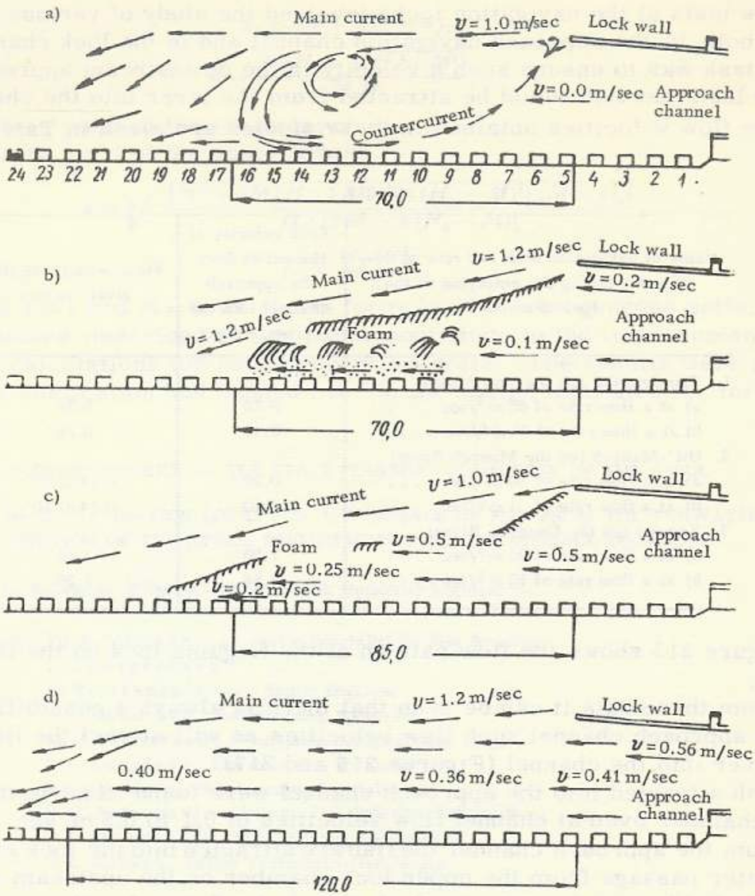


FIGURE 215. Flow pattern in the Daugava River near the approach channel of the lock

a–at closed lock chamber; b–at the passage of a 25 cm deep water flow through the upper lock gates; c–as (b) but for a 35 cm deep flow; d–as (b) but for a 60 cm deep flow.

Observations on the Tsimlyanskii and Ust'-Manych locks showed that in the downstream approach channel the water temperature in spring is by 2 to 3° higher than in the river; at the Kegums lock the water temperature during autumn is up to 0.5° higher than in the river (Table 3).

TABLE 3

Name of river	Water temperature (t °C)		
	in the river	in the channel	difference
Don River (May)	14.9	17.5	2.6
Manych River (May)	14.0	15.8	1.8
Daugava River (September)	17.7	18.2	0.5

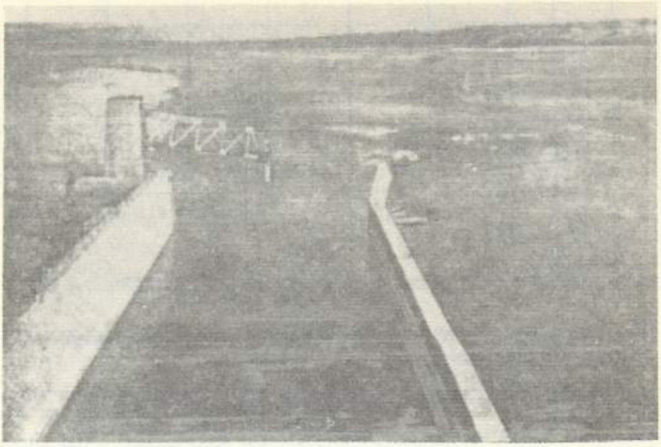


FIGURE 216. Approach channel for attracting fish from the river

FIGURE 217. Approach channel for attracting fish from the river

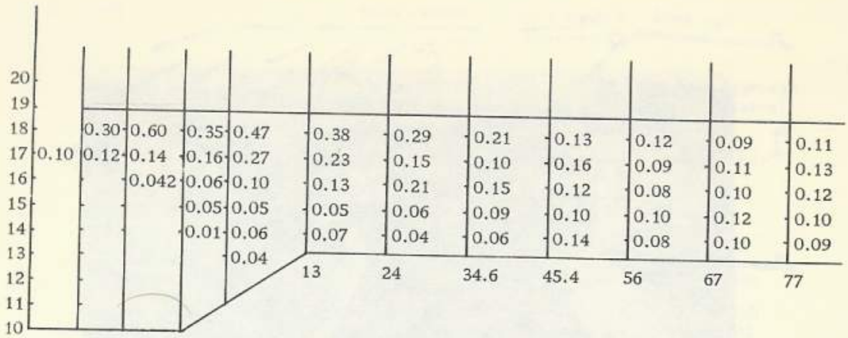


FIGURE 218. Streamflow distribution in the lock chamber during the passage of water for luring fish from the river into the channel

During the spring, the fish, swimming against the stream along the river bank adjoining the approach channel, will be attracted into the canal by warmer water currents.

Suitable water temperature and flow velocity in the downstream approach channel favored the attraction of fish into the lock chamber.

Ichtiological observations showed that all the fish varieties in the river enter the approach channel; most of them enter the lock chamber and pass upstream into the storage reservoir. This number of fish was checked by catching them with fixed nets which, however, were far from catching all fish entering the lock.

The results of the investigations and field tests proved the possibility of using low- and medium-head navigation locks for the passage of fish during the spawning periods.

The results of the experimental work carried out in 1957 were utilized in preparing the general design of the Stalingrad navigation channel, while the detailed plans made use of the results obtained during the 1958 experiments.

PART V

HYDRO CONSTRUCTION AND ITS ORGANIZATION

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR HYDRAULIC MECHANIZATION

Head: Professor M. A. Dement'ev, Doctor of Technical Sciences

FIELD INVESTIGATIONS DURING HYDRAULIC FILLING OF THE PAVLOVSK HEP DAM

Responsible for Research: V. A. Melent'ev, Candidate of Technical Sciences, Senior Research Worker

At the Pavlovsk HEP the diversity of rocks used for the hydraulic filling of the dam, and the method of filling – separate filling of the upstream and downstream dam shells and of the core with rocks from different quarries – necessitated special field investigations.

These investigations involved:

1) examination of the method of separate filling of dam shells;
2) study of flow of slurry over the hydraulic-fill slope;
3) measurements of filling rate in terms of height of filled layer.

The research led to the following conclusions:

1. The experience gained in separate filling of dam shells and core proved this method to be very efficient, as it widens the scope of hydro mechanization in hydro construction.

2. The gravel-pebble soil used for filling the upstream and the downstream dam shells, with respect to its petrographic composition, size, and shape of grains, is the least suitable of the soils used in the U. S. S. R. thus far. The granulometric composition of this type of soil (some boulders reach a size of 300 mm), sharp outlines of the grains, and their great hardness, causes rapid wear of the equipment.

3. Photographs of the flow of slurry over the hydraulic-fill slope showed that, at the entrance to the central settling pool, the ratio of the width of the slurry stream to the slope lenth was approximately 1:1.

The flow of slurry over the hydraulic-fill slope (see Figures 219, 220) can be described by the following relationship:

$$\frac{b}{B}=f\left(\frac{x}{L}\right),$$

where b = width of slurry stream;

B = width of slurry stream at its entrance into the settling pool;

x = distance between the slurry-pipe outlet and the filled layer;

L = length of filled slope.

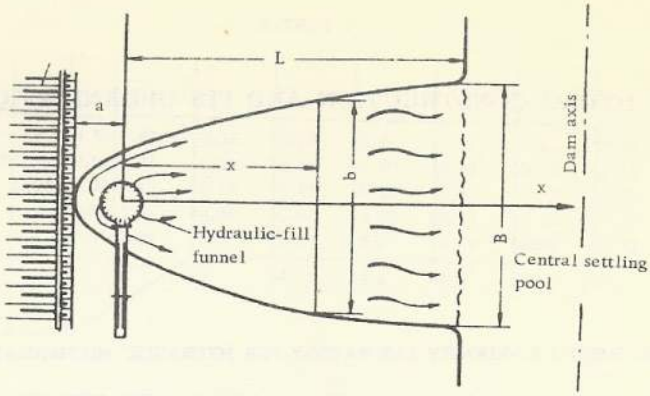


FIGURE 219. Schematic representation of slurry flow over the hydraulic-fill slope

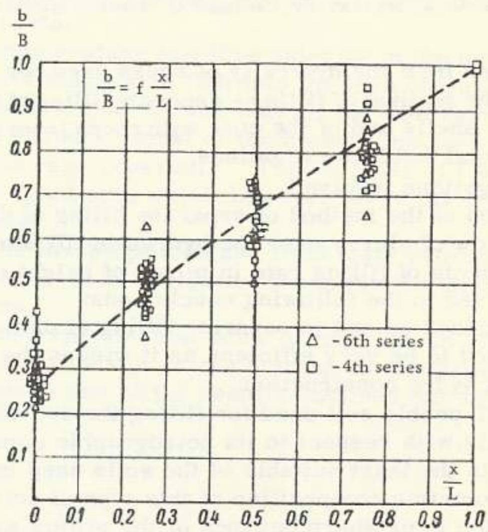


FIGURE 220. Flow of slurry over the hydraulic-fill slope

4. The actual filling rate of the dam layers at the Pavlovsk HEP was, on the average, 0.4 m/hr (in terms of dam height), while the over-all rate of erection of the dam was, on the average, 0.15 m/24 hrs.

The above investigations also involved granulometric and petrographic analyses of the fill material, photographic studies, brief description of the work carried out, etc.

The data obtained have been placed at the disposal of the design and construction organizations concerned.

LABORATORY RESEARCH OF DAMS FILLED WITH SANDY CLAY (LOAMY) SOIL

Responsible for Research: V. A. Melent'ev, Candidate of Technical Sciences, Senior Research Worker

Core-type earth dams are filled with a mixed soil: coarse-grained rock (gravel or sand) and cohesive rock (loam or clay).

Many problems of design and construction of core-type hydraulic-fill dams still await clarification. Among these are the following:

1) the character of the settling process of cohesive soil during the filling of the dam shells, and the effect of cohesive soil on the final physicomechanical properties of the filled layer of the shells;

2) the process of consolidation of the soil of the filled core-layers of the dam. Determination of the length of the transition period from the state of flowability (where the core, still a heavy liquid, exerts hydrostatic pressure on its lateral shells), the plastic and the solid state.

This work aimed at the study of these problems in the light of earth-dam construction in general, and of the dam of the Akstafa HEP in particular. It involved the following stages:

1. Study of the available literature and data on the design and construction of earth dams filled with cohesive soil.
2. Trial fillings, in order to study the effect of process technology and size grade of starting (quarry) rocks on the final physicomechanical properties of filled dam shells. Trial filling involves soils granulometrically close to the soils from the quarries found in the vicinity of the Akstafa HEP.
3. Study of the physicomechanical properties of cohesive quarry rocks, and of the formation of rounded-off lumps during excavation in the quarry, hydraulic haulage and filling.
4. Study of consolidation of the core soil on the basis of data obtained in drilling boreholes in the Yuzhno-Ural'skaya and Mingechaur earth-fill dams.

Results of investigations

The work described in this paper was actually a complex of experimental investigations carried out in order to find a proper basis for the use of cohesive soils in hydraulic filling. With respect to the grain-size distribution (granulometric composition) and technology of operations, the experimental hydraulic fillings represented typical cases likely to be encountered in practice. After data processing the following conclusions were drawn:

1. The physicomechanical properties of the hydraulic fill depend on the content of dispersive grains in the quarry rocks.
2. Within certain limits, the properties of the earth fill can be altered by varying certain technological factors, such as unit consumption of slurry deposited on the slope, and the rate of hydraulic filling.

Optimum filling conditions with a quarried material having grain sizes of < 0.05 mm, may be obtained by means of the so-called trestleless shuttle-type filling method involving deposition of the slurry over long slopes, and its discharge from the outlet end of the slurry pipeline.

3. The hydraulic filling experiments proved the possibility of filling the dam of the Akstafa HEP with a mixture of cohesive rocks and sand gravel.

4. The data on the physicomechanical properties of rounded-off lumps of cohesive soil permit a tentative evaluation of the suitability of these soils for "piece-type" filling.

5. Where the curve of the grain-size distribution of the soil of the dam core is within the standard range of boundary curves (Figure 221), this indicates that such a soil passes into the plastic state already during the construction period. For operational calculations of such soils there is no need to introduce the core pressure on the dam shells over the entire height with a side-pressure coefficient.

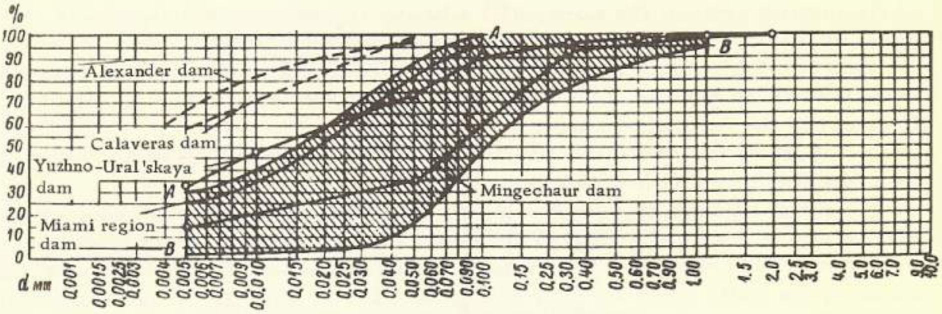


FIGURE 221. Core-type earth-fill dams

Figure 222 shows an example of variation of soil of a hydraulic fill-dam in terms of the relationship between the consistency factor and the layer depth for various stages of hydraulic filling of the Mingechaur HEP dam:

$$B = f(H),$$

where $B = \frac{W-A}{P}$;

W = percentage humidity (by weight);

A = limit of plasticity;

P = plasticity factor;

H = layer depth measured from the surface of the fill.

The report on the work carried out was handed over to the customer (Bakgidep) for use in the design of core-type hydraulic-fill dams.

The section of the report dealing with the study of consolidation of core soil was read at the Conference on Problems of Hydro Mechanization, organized by the NITO of the Building Industry in April 1959. Further, more detailed studies on the consolidation of soil in core-type hydraulic-fill dams, are needed.

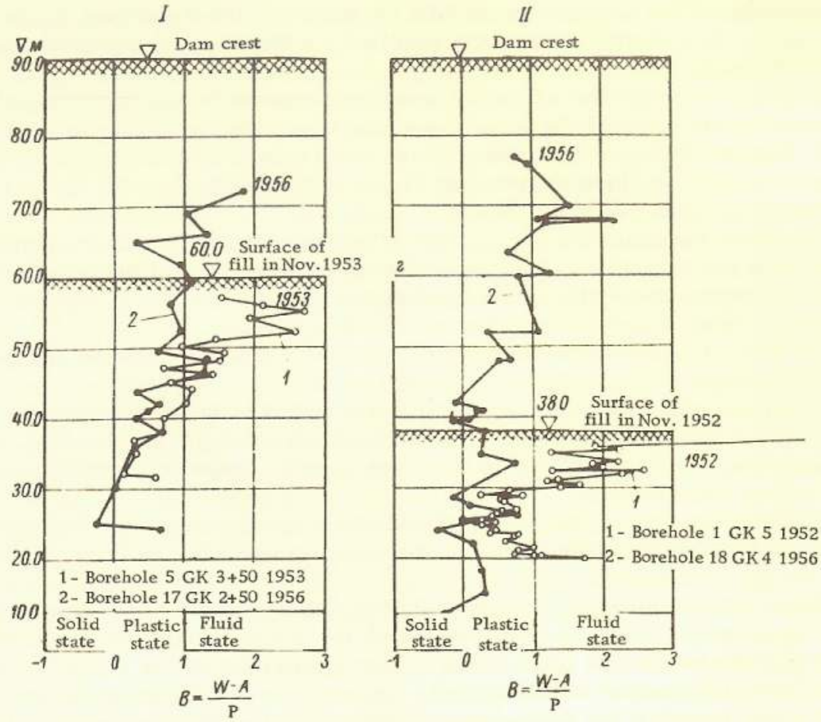


FIGURE 222. Graphs illustrating the state of the Mingechaur dam core

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR ORGANIZATION AND PLANNING OF HYDRO CONSTRUCTION

Head: I. E. Karklev, Candidate of Technical Sciences, Senior Research Worker

NEW TECHNOLOGICAL PROCESSES FOR CONSOLIDATION GROUTING OF ROCK FOUNDATIONS AND SOILS BY MEANS OF VIBRATION-GROUND AND HIGH -PLASTICITY CEMENT

Scientific Guidance: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

Research by: I. R. Arabadzhyan, Postgraduate

These investigations aimed at the study of a new, improved method of consolidation grouting by means of cement additionally ground in special vibration mills.

The investigations involved the study of conditions for obtainment of high-strength cement of reduced hardening time and improved physicomechanical

properties. This was accomplished by the addition of various surface-active agents increasing the cement plasticity without any further increase in the W/C ratio.

In order to study the effect of grinding fineness on the basic physicomechanical properties of the grouting mixes and of the cement stone obtained from the hardening of the latter, the cement was ground in a vibration-type cement mill to various degrees of dispersion, measured with the aid of the Deryagin and the PSKh-2 apparatus.

The investigations on the method of consolidating the sandy alluvial soil by grouts prepared from ground cements covered the following stages:

1) study of properties of consolidation grouts and of the cement stone prepared from wet-or dry-ground cement;

2) consolidation of water-saturated, sandy alluvial soils by means of cement grouts.

The properties of cement grouts were tested at W/C ratios of 0.45; 0.7, 1.0 and 2.0, and for durations of additional cement grinding of 3, 5, 7 and 10 minutes, respectively. The cements used in these tests varied in mineralogical composition.

In order to assess the effect of additives, spent wash from sulfide-cellulose processes (SW) was added to the grout in amounts of from 0.15 to 2.0% (by weight of cement).

After 3 minutes of additional grinding, the plasticity of the cement grout was found to increase as compared with the initial (starting) plasticity. This may be explained by the fact that comminution of the large particles and flakes (floccules) existing in the cement powder leads to the reduction of friction between the particles and, hence, to greater plasticity of the grout.

As the duration of additional cement-grinding increases, the plasticity of the grout mix decreases. The reduction, however, is less drastic with wet grinding than in the case of dry grinding, apparently due to the greater degree of dispersion attained by the wet process in the same grinding time.

Both dry and wet grinding increase the resistance of the grout to stratification by enhancing the degree of dispersion of cement particles and by raising the content of particles of colloidal dimensions. Addition of SW leads to a further increase in resistance to stratification, owing to the peptinizing effect of this additive.

The strength of the cement stone based on dry-ground cement exceeds that of wet-ground cement, particularly at high W/C ratios. This may be due to an unsatisfactory ratio of the gel to the crystalline component of the cement stone, since during wet grinding, the cement particles undergo intense hydration which leads to the formation of large amounts of gel.

The compactness of dry-ground cement stone is higher than that of wet-ground cement. Increased porosity of the latter is due to the entrapment of an increased amount of air and water during the grinding process.

These investigations proved the advantage of dry-ground over wet-ground cements.

Watertightness tests of the cement stone revealed higher watertightness of additionally ground cement as compared with cement used without additional grinding.

In order to study the effect of inert micro-aggregates (fillers) on the mechanical properties of the cement stone structure, ground quartz sand was

added to grout prepared from additionally ground cement to form a cement-to-sand ratio of 1:1.

Although addition of inert micro-aggregates lowered the plasticity of the mixed grouts, plasticity of various grout mixes differed but little. For example, the degree of spreading of a pure cement grout made of a 3-minute dry-ground cement produced by the Volkhov mill, at the W/C ratio of 0.7, was 121 mm; the addition of micro-aggregates lowered this value to 118 mm. At a W/C ratio of 1.0, and a 5-minute duration of additional grinding, the degree of spreading increased to 138 mm and 136 mm, respectively.

Addition of SW plastifiers (0.1% by weight of the mix), and of combined plastifiers (0.1% SW plus 0.01% neutralized wood pitch) to grouts containing ground micro-fillers, improves plasticity and fluidity of the grout.

Data on the adhesion bond between cement grouts and various rocks and concrete show that the chemical and mineralogical composition of rocks has a strong bearing on the shear and breaking strength of the bonds.

Experiments carried out by the VNIIG on grout consolidation of sandy water-saturated soils proved the efficiency of additionally ground cement grouts within the spreading range of 180 cm.

The grout-consolidated sandy soil has high strength and watertightness. For example, after 28 days of hardening, its strength was 50 to 76 kg/cm^2, and its seepage rate decreased by a factor of from 80 to 1800.

High permeability of vibration-ground cement grouts makes it possible to:

a) lower the W/C ratio to between 1 and 3 (i.e. by a factor of 3 to 10), and thus to increase the compactness and strength of the consolidated soil;

b) reduce the injection pressure of grouting;

c) increase the action range of grouting, and thus to increase the distance between the grouting boreholes up to 3.5 m in construction of seepage-preventing grout curtains.

Experiments with consolidation of sandy soils having a seepage rate of 50 to 52 m/24 hrs, showed the possiblity of effective consolidation within an action range of grouting varying from 125 to 155 cm.

HIGH-PRESSURE INJECTION OF CEMENT-CLAY GROUTS INTO ALLUVIAL SOILS

Responsible for Research: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

Research by: L. N. Paronyan, Junior Research Worker

The Churubai-Nura dam, whose foundation rests on alluvial bedrock, had to be protected against seepage by means of a special curtain 28 m deep. Having recommended the construction of a consolidation grout curtain, VNIIG carried out its design, while delegating the conduct of planned field investigations to the Gidrospetstroi Trust.

VNIIG also participated in 1957 in the analysis of the investigation results directly on the construction site. In 1958 the Institute tested selected specimens of grouted soil and prepared the final report on the test results.

Tests were carried out with cement-clay grouts having the following cement-clay-bentonite-water ratio: 1 : 2.83 : 0.5 : 12.3 (by weight).

The Karaganda branch of Gidrospetstroi sent to the VNIIG three batches of grouted soil specimens.

These specimens, tested for strength and watertightness, may be classified in three groups:

1. Specimens of very low strength, which were tested mainly for watertightness. The specimens contained a great amount of clay and very little cement. Their watertightness corresponds to a seepate rate of 10^{-3} cm/sec.

2. This group comprised specimens tested not only for watertightness but also for strength. Their strength varied from 5 to 8 kg/cm^2; their watertightness was characterized by seepage rates varying from 10^{-5} to 10^{-6} cm/sec. These specimens had compact cement-clay interlayers 0.5 to 1.5 cm thick.

3. The third group consisted of more resistant and compact specimens (10-15 kg/cm^2); their watertightness had quite the same values as that of the specimens of the second group.

Apart from the above tests, the specimens were also tested for macerability and porosity.

It should be noted that the report on the tests sent to the institute indicated only the depth of sampling, while no data on location of the specimens in plane and on their distances from the grouting holes were available. This did not permit full assessment of the quality of test grouting.

Had the tests been carried out under the prescribed conditions, the quality of grouting could have been markedly improved. No improvement, however, could be expected when, as was the case, groutings of nearly the same composition were injected into alluvial soils of different grain-size distribution.

Soils composed of gravel and coarse-grained sand, which should have been grouted with cement-clay-sand grouts, were in fact grouted only with cement-clay grout. Furthermore, the grouts contained bentonites which, if present in large amounts, markedly reduce the quality of the resulting cement stone. There were other infringements of the prescribed testing-technique procedure which adversely affected the grouting quality. In cases where the soil exhibits marked absorption of the grout mix, grouting should be done intermittently with gradual reduction of grouting pressure.

A BASIC OUTLINE FOR THE PLAN OF GROUTING THE CONCRETE CONSTRUCTION JOINTS OF THE BRATSK HEP DAM, AND RECOMMENDATIONS FOR ITS EFFICIENT IMPLEMENTATION

Responsible for Research: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

This study involved: 1) compilation of a basic outline for the plan of grouting the concrete construction joints of the Bratsk HEP dam; 2) the drawing up of technical specifications for quality control of grouting; and 3) complex laboratory tests for the determination of appropriate grouting mixes suitable for use at subzero temperatures.

The basic outline of the plan for grouting construction joints was drawn up and submitted to the customer. The outline contained new suggestions of grouting with the aid of inflatable rubber hoses inserted into the concrete massive and withdrawn after its hardening.

The report also includes a bibliographical survey on grouting of construction joints at various foreign dams. Finally, it contains instructions on quality control of grouted construction joints.

RECOMMENDATIONS AND TECHNICAL SPECIFICATIONS FOR THE DESIGN OF GROUT CURTAINS AND FOR CONSOLIDATION GROUTING AT THE CHARVAK HYDRO DEVELOPMENT

Responsible for Research: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

In 1958 the final report relating to the design of the grout curtain for the foundations of the Charvak HEP dam, and regarding development of suitable grout-mix compositions, was completed. The report also contains brief technical specifications applicable to the geological and hydrogeological rock characteristics of the foundation concerned. The conclusions relating to the design of the grout curtains envisage wide use of cement-clay grouts mixed with plastifying additives.

MEASURES ENVISAGING IMPROVEMENT IN ORGANIZATION OF WORK AND REDUCTION OF COSTS OF EARTHMOVING IN HYDRO CONSTRUCTION

Responsible for Research: V. N. Lofitskii, Candidate of Technical Sciences, Senior Research Worker

Research by: M. A. Shkabara, Senior Engineer

The aim of the present study was: 1) to ascertain the advisability of using railroads for transportation of earth and rock excavated from construction sites of hydro developments; 2) to devise improved methods of fighting ice and freezing of soil during excavation by means of floating suction dredges.

Study of Soviet and foreign literature on the subject as well as the personal knowledge of the authors served as the basis for solving these problems.

With respect to point(1) the researchers arrived at the following conclusions:

1. For excavations having a depth greater than 30 m and a volume of earthwork above 2,000,000 m^3, railroad transportation of the excavated material is economically suitable: it was found that in many cases railroad transportation is cheaper than truck transportation (sometimes by $\sim 30\%$).

2. The experience gained by the Soviet mining industry in the use of railroad facilities should be kept in mind when studying the use of railroad facilities for transportation of earth and rock from the construction sites of hydro projects.

With respect to point (2) the researchers:

1) prepared performance characteristics of the suction dredges used in winter excavations that involved both submerged and nonsubmerged excavation;

2) prepared recommendations for redesigning the existing Soviet-made suction dredges in order to adapt them to operation at subzero temperatures;

3) carried out heat-engineering calculations applicable to the modern methods of keeping portions of the river free of ice under conditions of permanent freezing and ice formation;

4) worked out new methods for keeping portions of the river free of ice;

5) studied conditions of slurry flow through pipelines and prepared recommendations for operating such pipelines in winter;

6) studied various methods of hydraulic filling of earth structures in winter and suggested new hydraulic-fill methods.

The enormous volume of earthwork planned for the forthcoming Seven-Year Plan calls for continual study of operation of single-bucket and multi-bucket suction dredges in order to raise the efficiency of hydromechanical work under winter conditions.

VNIIG IMENI B. E. VEDENEEV ICE-THERMAL LABORATORY

Head: Professor A. M. Estifeev

THERMAL CALCULATION OF ARTIFICIAL COOLING OF CONCRETE BLOCKS AT THE BRATSK HEP

Responsible for Research: A. I. Pekhovich, Candidate of Technical Sciences, Senior Research Worker

Research Team: S. M. Aleinikov, Junior Research Worker
S. I. Artem'eva, Senior Engineer

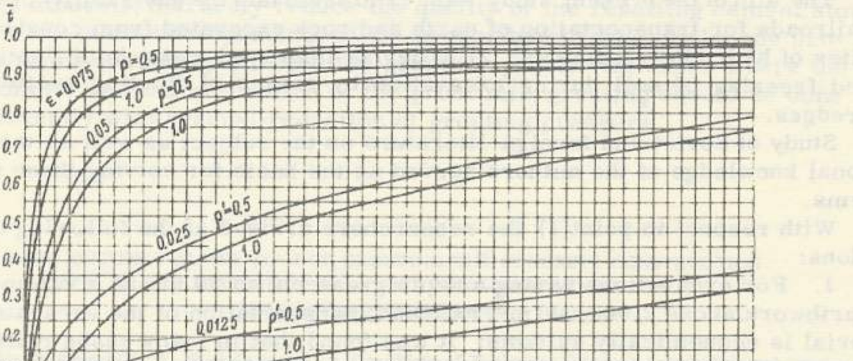


FIGURE 223. Chart for calculating $\bar{t} = f(\rho', F_0, \varepsilon)$

This study dealt with new methods of thermal calculation of artificial cooling, particularly as it applies to the core of concrete blocks. The study involved the determination of temperature distribution in the concrete

surrounding the coolant-carrying tube as well as determination of heat-content variations in the concrete block being cooled.

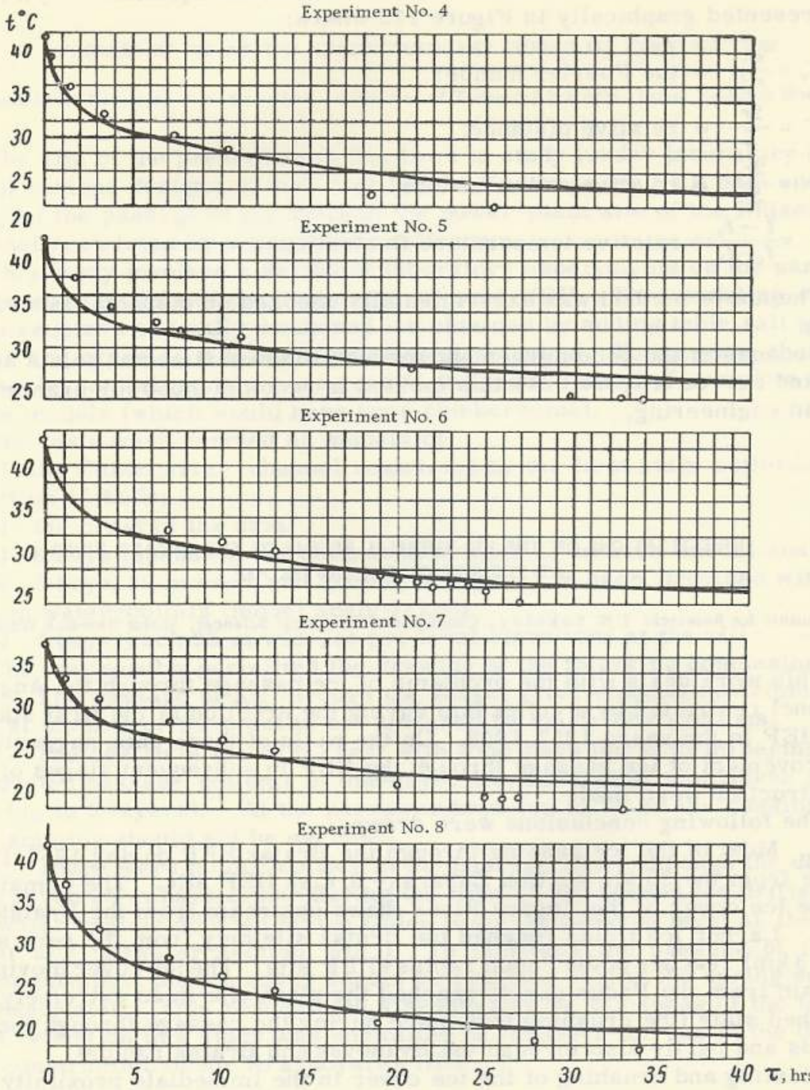


FIGURE 224. Variation of temperature with time, as measured by thermocouple No. 2 (calculated curves and experimental points)

Preliminary theoretical studies enabled the staff to devise a tentative method for calculating variations of temperature (with time) in a concrete block cooled by a constant-temperature coolant flowing through tubes laid in the block.

The above method permits the determination of variations of temperature (t) with time (τ) at various points (r) of the block, as a function of tube radius (r_0), distances between tubes (S), coolant temperature (t_c), initial temperature of the block (t_0) and coefficient of coolant diffusivity of concrete (a).

In order to facilitate calculations, the analytical equation $\bar{t} = f(F_0, \rho', \varepsilon)$ is presented graphically in Figure 223 where:

$F_0 = \frac{a\tau}{r_0^2}$ = the Fourier number;

$\rho' = \frac{2r}{S}$ = relative distance;

$\varepsilon = \frac{2r_0}{S'}$ = relative radius of tube;

$\bar{t} = \frac{t - t_0}{t_c - t_0}$ = relative temperature fluctuations.

The above method was experimentally checked on a special laboratory setup.

Because of the good agreement between the experimental points and calculated curves (Figure 224), this method is recommended for practical use in engineering.

INVESTIGATIONS OF THE ICE-THERMAL REGIME OF THE ANGARA RIVER AT THE SITE OF THE BRATSK HEP

Responsible for Research: I. N. Sokolov, Candidate of Technical Sciences, Senior Research Worker

This work deals with the problems of ice passage through the Angara channel restricted by a cofferdam during the erection of the first stage of the HEP in the years 1957, 1958. On the basis of these data, suggestions for improvement of ice passage through the HEP in subsequent stages of its construction were made.

The following conclusions were drawn:

1. Most of the ice passing through the Bratsk HEP during 1957/1958 came from the 30 km section Zayarsk–Bratsk HEP site. The remainder of the ice cover of the Angara River thaws upstream from the Bratsk HEP.

2. Larger ice fields reached the Bratsk site only from the very short (2 to 3 km) reaches of the Padun rapids–HEP site. The ice cover moving downstream from the Padun rapids reached the plant site in an extremely crushed state (the crushing took place during the passage through the Padun rapids and partly also through the Pyanovsk and Bratsk rapids).

Blasting and crushing of the ice cover in the immediate proximity of the plant site (from the Padun rapids to the plant site, and also downstream from the plant) yielded good results because it eliminated excessive water retention in the tailwater section and prevented bottlenecks at the inlet into the channel section restricted by the cofferdam. These measures are, therefore, recommended for future use.

In order to facilitate the passage of ice through the temporary cofferdam, which will be erected during the second stage of construction, it is necessary: 1) to plan measures for breaking up large ice fields upstream from the Padun

rapids which by then will be submerged and cease to be, as it were, a crushing mill; 2) to dismantle or, if impossible, to properly fasten the temporary concrete-conveying trestles installed at lower elevations.

PASSAGE OF ICE AT THE KRASNOYARSK HEP DURING ITS CONSTRUCTION

Responsible for Research: I. N. Sokolov, Candidate of Technical Sciences, Senior Research Worker

The aim of the present work has been to study (under laboratory conditions) various designs of the crest for the Krasnoyarsk HEP dam and the study of the passage of ice through the power-plant site of the Enisei river channel restricted by a temporary cofferdam.

This study involved a series of laboratory experiments on the passage of ice through models of the dam crest of said HEP. The modeling conditions required specially weakened ice obtained by adding table salt to the water and exposing it to the light of electric bulbs. To increase the modeling scale, partial models were used in the tests instead of complete large-scale models (which would have been cumbersome).

The tests were carried on models of:

1) the Enisei river channel restricted by the first-stage cofferdam (restriction of 60%);

2) the crest of the dam:

a) 6 bays (spans) of the dam, each of them 11 m wide (model scale 1:60);

b) 2 bays, 39 m each, and another 4 bays, 11 m each, provided with two bottom water-outlets (model scale 1:100);

c) 4 bays, each 25 m wide, at two level elevations at the sill.

The test results permitted the drawing of the following conclusions:

1. The passage of ice through the Enisei river channel restricted by 60% at the site of the plant is not hindered by major obstructions.

2. The design of the dam crest with wide bays (up to 39 m) permits unrestricted passage, not only of one-layer ice, but also of multilayer ice covers (up to 3 layers). At the narrower bays (up to 11 m) the possibility of ice jamming should not be excluded.

3. There were no difficulties in the passage of ice through the dam crest with seven 25 m bays, either during the first or the second ice drifts, under the water regime adopted by the Gidep (water-discharge level at the headwater and tailwater section, etc.). To facilitate the passage of ice as far as possible, the ice cover should be retained during the spring season at the headwater. This reduces the strength and the thickness of the ice cover.

A report on this study was submitted to Lengidep for use in the design and construction of the Krasnoyarsk HEP.

ORGENERGOSTROI MSES LABORATORY OF AUTOMATION

Head: Yu. G. Miller, Engineer

AUTOMATIC RECORDERS OF PERFORMANCE OF STRAIGHT-SHOVEL EXCAVATORS AND OF DUMP CARS

General Guidance: Yu. G. Miller, Engineer
Professor L. F. Kulikovskii, Doctor of Technical Sciences

Research Team: G. V. Bozin, Engineer, Head of Research Team
V. Ya. Ratnovskii, Engineer
L. A. Brovkin, Engineer
A. M. Lar'kov, Engineer

At present, operational performance of excavators and of dump cars is recorded by registering the number of dump cars used for transportation of excavated ground, and the number of operation cycles of the excavators.

This work is usually done by special workers, who enter into a log book all the data on the performance of excavators and of dump cars. This method, however, is inefficient, inaccurate, and expensive. A method of automatic recording is urgently needed.

This is a description of studies carried out by the above laboratory on automatic recording of the performance of straight-shovel excavators and of dump cars.

The paper deals with previous studies on automatic recording of performance of excavators and dump trucks, and presents a critical review of the various recording devices described therein.

The possible solutions of the problem of automatic recording are analyzed and the most advantageous of them selected. Methods used in designing elements of recording devices are described. Particular attention is paid to the selection of primary (sensing) elements (i.e. weight-measuring elements), their arrangement within the device, and to the question of their reliable operation. The study involved manufacture of experimental models and their field-testing on the E-1003 excavator and the ZIL-585 dump truck at the site of the Kuibyshev hydro development. The results of tests showed that, although the automatic recording devices of the ER-1 type and the RAS-1 type are operationally reliable, they need to be further improved both in design and performance.

The use of these recorders will permit a 70% reduction in accounting expenses, improvement in accuracy of recording the volume of earthwork, greater serviceability of the equipment, and a high level of handling and maintenance.

An appendix to the paper contains drawings of the basic subassemblies of the ER-1 and RAS-1 automatic recorders.

ORGENERGOSTROI MSES DEPARTMENT FOR NEW INDUSTRIAL PROJECTS

Head: V. B. Mints, Engineer

REINFORCED PANEL FRAMEWORKS AND BLOCKS FOR CONCRETE AND REINFORCED-CONCRETE HYDRAULIC STRUCTURES

Responsible for Research: V. B. Mints, Engineer

Research Team: G. Ya. Alasyuk, Senior Engineer
A. E. Novitskii, Senior Engineer
E. E. Kucheryavenko, Senior Engineer
Yu. N. Buslaev, Engineer

The purpose of this paper was:

1) to study the experience gained in the use of reinforced panel structures in hydro construction;

2) to select the best designs of reinforced panels for different elements of hydro structures;

3) to study problems of combined use of reinforced panels and monolithic concrete blocks;

4) to design and to check under field conditions new technological processes for manufacture of reinforced panel structural elements, and to study the question of their use in the erection of large hydro structures;

5) to study the effectiveness of large-scale use of reinforced panels in hydro construction.

Reinforced panels combine in a single element both formwork sheathing and working reinforcement. They are free from many of the shortcomings inherent in structural elements made of separate reinforced frameworks and facing slabs. The main shortcomings of the latter are the excessive use of reinforcement to ensure rigidity of the framework during its transportation and erection, and insufficient use of the reinforcement of facing slabs incorporated into the structure. Another disadvantage is the difficulty of placing concrete by means of troughs, owing to large amounts of reinforcement in frameworks, and of ties for fastening the facing slabs. Finally there is the insufficient use of the load-carrying capacity of cranes used in concreting blocks, etc.

Experience gained in the use of reinforced panel structures in the construction of the Kuibyshev and Stalingrad hydro plant, brought out two particularly interesting types of reinforced panels. The distinguishing feature of the first type is the arrangement of the basic reinforcing element within the slab of the panel, and that of the second type – the location of the basic reinforcing element outside the slab. In the second type, reinforcement is fastened to the slab by means of undulated reinforcing rods.

The paper describes both types of reinforced panels, each weighing up to 20 t, and the possibilities of their use in all the various reinforced-concrete structural elements.

Investigations of the behavior of combined reinforced panel and monolithic concrete blocks confirmed the initial assumptions of the designers and of the available data on the subject. The various types of fastening and anchoring elements, such as the undulated reinforcing bars, the ties, the

box-type panel surface and the common "roughened" plates, when acted upon by operational load and by water pressure, did not exhibit any traces of shear in the joint or detachment of the reinforced panel from the supporting monolithic concrete block.

Experimental results so far confirm the reliability, the efficiency of design, and the economical effectiveness of reinforced panel elements as compared with structures built of reinforced frameworks and facing slabs. These advantages are as follows:

1) reduction in labor expenditure ~ 30%;
2) savings in metal ~ 20%;
3) savings in cement ~ 20%;
4) reduction in cost of 1 m^3 of placed concrete by 11%;
5) reduction in construction time by approximately 30 to 35%.

The reinforced panel elements designed by the department personnel were in part adopted at the Stalingrad HEP in the construction of the pier for the 22nd turbine set. The panels were made with protruding working reinforcement.

All of the design work was carried out by the power-plant section of the Gidroproekt.

ORGENERGOSTROI MSES DEPARTMENT FOR PLANNING AND MECHANIZATION OF EARTHMOVING WORK

Head: S. I. Dudintsev, Engineer

METHODS OF MEASURING VOLUMES AND AREAS WITH THE AID OF GROUND STEREOPHOTOGRAPHY

Responsible for Research: B. S. Puzanov, Candidate of Technical Sciences
N. I. Ivanov, Engineer

Because construction of large hydro projects involves displacement of huge volumes of earth, accurate measurements of the volume of earthwork is of particular technical and economical importance. This study deals with the question of replacing the existing methods of measuring volumes with the aid of topographic maps or profiles by the more efficient procedures of ground stereophotography.

The paper describes the theoretical principles of direct volume and area measurements by ground stereophotography using horizontal and vertical coordination grids* without the use of maps or profiles.

This method involves the use of simple and cheap equipment – of phototheodolites of any design and of stereophotogrammetric apparatus.

The over-all working time for measuring volumes by the new technique is only 1/4 to 1/5 of the time taken by conventional geodetic procedures. Moreover, measurement accuracy is increased due to elimination of errors in graphical plotting

* [See "Elements of Surveying", Department of the U. S. A. Army, Oct. 1953, page 137.]

The efficiency of the method can be improved by attaching a computing device to the sterophotogrammetric unit, and by improving the design of the phototheodolite. This will permit the taking of daily measurements.

The method was successfully applied at the sand and rock pits of Kuibyshevstroi; at present it is being adopted at the construction site of the Bratsk HEP.

MECHANIZATION OF DRAINAGE WORK (DRAIN PLACING) IN HYDRO CONSTRUCTION, AND ECONOMICAL DESIGN OF VERTICAL AND HORIZONTAL DRAINS BASED ON EXPERIENCE IN OPERATION OF DRAINAGE SYSTEMS

Responsible for Research: F. F. Fokin, Engineer

Research Team: Yu. M. Anan'ev, Engineer
Yu. P. Buldakov, Engineer

The paper mainly deals with the design of drainage systems for earth dams and their mechanical placing.

Drainage of earth dams is not only important, but, in many cases, mandatory. The cost of drainage work - one of the basic elements of hydro construction - amounts to 10-30 % of the over-all cost of the earth dam.

Drainage systems which have been used so far in earth-dam construction are quite complicated and are not devoid of a number of shortcomings.

In order to reduce the amount of drainage work and to obviate the use of manual labor, emphasis was placed on the study of the problems of design and on complete mechanization of drainage works.

The paper gives a brief outline of the theory and the methods used in the design of drains and filters, and their placing in earth-fill dams. It describes the operation of completed drainage systems, cites the results of observations of the behavior of the seepage line in the dam body, and describes the operation of drainage systems in the Tsimlyanskii, Kakhovka, Kuibyshev, Stalingrad and, partly, in the Gor'kii HEPs.

Data are given on foreign experience in river damming and drainage placing. They refer to the Genissiat dam on the Rhone (France), Fort Peck and Fort Randall on the Missouri (U. S. A.).

The problem is studied of designing economic drainage systems for earth-fill dams, particularly of drainage systems produced on a large-scale industrial basis.

The paper describes the following types of drainage systems used in practice:

a) rock blanket with reverse filter;

b) rock toe in front of the rock blanket;

c) temporarily installed slabs and reinforced-concrete perforated pipes;

d) reinforced-concrete drainage main with spongy-concrete filtering core;

e) spongy-concrete pipes;

f) with riprap layer.

In order to find optimum design and operation conditions, the staff carried out special hydraulic, static and seepage calculations.

The most economic design of drainage systems proved to be the one consisting of spongy-concrete pipes with coarse-grained sand filler. This design permits both industrial-scale manufacture and mechanized placing.

It is assumed that this type of drainage will cost from 20 to 30% less than the drainage systems of the Gor'kii and the Kuibyshev HEPs.

Although so far the methods described in this paper have not been applied, they are applicable for construction of the Votkinsk, Kaunas, and Cheboksary HEPs.

DECISIONS ON EARTHWORK PLANNING IN THE CONSTRUCTION OF HYDROELECTRIC POWER PLANTS AND ELABORATION OF TECHNOLOGICAL CHARTS AND REGULATIONS

Responsible for Research: S. I. Dudintsev, Engineer

Research Team: B. M. Anikeev, Engineer
V.E. Chubov, Engineer
N. G. Rzhevskii, Engineer
A. N. Putenikin, Engineer
Kh. G. Tukhvatullin, Engineer
Yu. S. Trunin, Engineer
I. A. Katin, Engineer, and others

The paper gives a detailed description of problems of all-round mechanization of such common types of earthwork as: excavation of canals, trenches, erection of dams and weirs with the use of excavators, dump trucks, tractor-hauled scrapers, and hydro-mechanical equipment, consolidation of slopes by concrete slabs, and driving-in and pulling-out of metal piles by the vibration-impact technique.

The study amply draws on the experience gained in the construction of such advanced hydro projects as the Volga-Don Canal, the Kuibyshev, Gor'kii, Kakhovka, and Stalingrad HEPs, as well as on the previous studies carried out by the various scientific research and design institutes: NIIOMS, SoyuzdorNII, Gidroenergoproekt, Spetsstroiproekt, etc.

This work examines various technological charts and schemes of all-round mechanization, which is invariably based on the use of Soviet-produced equipment most widely used in construction. Equipment is selected with the view of ensuring full mechanization of the basic, most labor-consuming processes.

Brief instructions on the planning of earthwork by various methods are presented along with technical and economical characteristics, calculated according to the average performance of the technically most advanced construction sites. These characteristics may be applied to the design and planning of earthwork, together with the recommended technological charts and regulations developed in this work.

Application of the latter in the construction of the Votkinsk HEP and the Dnieper-Krivoi Rog Canal yielded the following results:

a) average productivity of excavators per set of machines [excavator, dump trucks and auxiliary equipment] increased by 76 % over the norms used hitherto; expenses per shift were reduced by 44 %;

b) the productivity of scrapers increased by 43%; labor expenditure decreased by 38%;

c) it is anticipated that savings through the reduction of cost of excavator and scraper operation will amount to 18%.

The scientific-technical report prepared by the staff consists of the following sections:

I – Standard technological charts for earthwork operations employing excavators and dump trucks.

II – Idem for scrapers.

III – Idem for hydro-mechanized equipment.

IV – Idem for slope leveling and slope consolidation by means of reinforced-concrete slabs.

V – Methods of loosening frozen soil.

VI, VII, VIII – Standard technological charts for pile-driving work carried out with the aid of impact and vibration-type equipment (for driving-in and extracting sheet piling).

NEW DESIGN METHODS FOR CONSOLIDATING ROCK BLANKETS OF RIVER-DAM FOUNDATIONS

Responsible for Research: A. S. Paevskii, Engineer

The paper describes a new empirical method for the design of rock blankets on artificially consolidated or on solid-rock foundations of large HEP river dams.

The calculation formulas are based on the comparison of the results of experimental investigations with the results of field observations carried out by Professor S. V. Izbash.

The investigations were conducted on models at the hydraulic laboratory of theMoscow Power Engineering Institute, the Stalingrad hydraulic laboratory of the Orgenergostroi and the Kuibyshev Civil Engineering Institute.

A special chapter deals with the results of experiments carried out to determine the hydraulic size [fall velocity] of rock particles.

The formulas derived by the author make it possible to determine with sufficient accuracy the amount of riprap, rock, and concrete required for river damming under conditions of uninterrupted water flow.

The method ensures reduction of waste materials used for river damming and thus permits large savings of State funds.

WAYS OF REDUCING THE COST OF SEEPAGE-PREVENTING WORK IN HYDRO CONSTRUCTION

Scientific Guidance: A. V. Artem'ev, Candidate of Geological and Mineralogical Sciences

Research Team: J. V. Agafonov, Engineer
P. M. Bespalov, Engineer
G. A. Rodionov, Engineer

The paper described the experience gained in design, construction and operation of seepage-preventing structures at the construction site of the Tsimlyanskii, Kakhovka, Gor'kii, Volga imeni Lenin, and Stalingrad HEPs.

Technical and economical calculations permitted the design of efficient seepage-preventing systems, ensured a 35% cost reduction of construction work, and served as guiding posts for efficient design, construction, and operation of such systems.

From experience gained at the construction sites, the authors arrived at the following conclusions:

1. The proper design of seepage-preventing structures requires scientific selection of rated values and exact methods of calculating seepage, which should take into account the structural and hydro-geological characteristics of the area of excavation and of the foundations for the hydro structures.

2. For the correct calculation of the seepage-water inflow into the water-level-reduction wells, it is necessary to exclude from the over-all water seepage the amount of seepage water entrapped by the excavator bucket, by seepage grout curtains,diaphragms,etc.

3. Calculations should be based on the dry-weather period and on conditions of optimum load on the pumps. The amount of standby wells and pumping equipment required for the high-water season should be determined on the basis of conditions of limiting load on the pumps, optimum open-channel drainage conditions,and installation of lightweight wellpoints, taking into account the probable quality of the seepage-preventing sheet piling mounted in the body and the foundation of the river dam. The distance between the wells should be calculated on the basis of conditions of maximum delivery at full load on the pumps and the electric motors.

4. The suitable location of drainage wells and of pump sets markedly reduces the required number of deep drainage wells, the over-all amount of construction work, and raises the efficiency of utilization of the powerful pump sets by a factor of 1.5 to 2.

5. In order to speed up drilling of drainage wells – one of the basic elements of construction work – it is recommended to use, apart from the common percussion-cable drilling, also other special techniques for rapid drilling such as hydraulic, hydraulic-vibration, rotary, etc. The report also indicates the field of use for each of these methods.

6. In order to increase the draining capacity of filters and to save expensive steel pipes, it is suggested to use frame-type, polyvinyl, asbestos-cement, block-type filters made of porous concrete, etc.

7. Apart from the artesian-well turbine pumps, pumps with small-size submerged electric motors should be used. The type range of pumps should be extended.

8 The paper assigns fields of application for the ejector-type wellpoints, small-size wellpoint units, and for electroosmotic drainage. For

shifty soils of low permeability, permafrost curtains are recommended, in addition to electroosmotic drainage.

9. The work furnishes basic recommendations for automation of drainage systems, automatic measurement of water level, discharge, and of power consumption.

10. The paper formulates the basic principles for establishing new norms and engineering specifications for the design and construction of drainage works and for the drawing up of higher operational norms. It is suggested to use 1 m^3 as a unit for measurements and calculations of volume and cost of drained-off water. The degree of infiltration and percolation of various soils must be taken into account in the above calculations.

The proposals are expected to yield considerable economic results.

SCIENTIFIC RESEARCH INSTITUTE ORGENERGOSTROI MSES DEPARTMENT FOR PLANNING AND MECHANIZATION OF CONCRETE WORKS

Head: K. F. Kurnosenko

ALL-ROUND MECHANIZATION OF CONCRETE PREPARATION AND PLACING WITH THE AID OF TECHNOLOGICAL OPERATIONAL CHARTS FOR LARGE AND MEDIUM-SIZE HYDRO CONSTRUCTION

Responsible for Research: N. F. Edunov, Engineer

Research Team: L. I. Vlasov, Senior Engineer
S. I. Varzhov, Senior Engineer
V. G. Bogdanov, Engineer
E. P. Varzhova, Engineer
V. P. Tkachev, Engineer
V. S. Sidorov, Engineer

On the basis of experience gained in concrete preparation and placing in the construction of the Kuibyshev, Stalingrad, Kairak-Kum, Novosibirsk, and Bukhtarma HEPs, the present study worked out standard technological charts of all-round mechanization.

Stage 1 – gathering data on concrete preparation and placing at the construction sites, and compilation of separate technical reports for each site;

Stage 2 – economical and technological estimation of concrete work, and selection of suitable schemes for standard technological charts;

Stage 3 – final drafting of the standard mechanization charts.

The paper recommends the use of 19 standard technological schemes for various elements of basic hydro structures and mechanization equipment.

Each scheme has been adapted to the whole range of concrete work: installation of reinforcement, mounting and dismantling of formwork, placing and curing of concrete, etc.

All technological patterns contain a description of the engineering materials and labor required; they also furnish the basic technical and economical characteristics. Each scheme is supplemented by a technological chart for each type of concrete work: reinforcement, formwork, and concrete placing.

For each type of work the sheets contain the following data:

a) instructions on the use of the sheet;
b) basic characteristics of the process;
c) operational layout;
d) organization of labor;
e) operational instructions;
f) instructions on labor safety;
g) requirements of materials and machines;
h) manpower requirements;
i) work schedule;
j) technical and economic parameters of the whole process.

A special appendix to the charts gives instructions on the work performance during the winter.

The use of technological charts ensures precise work, facilitates the task of supervisers and of formen by freeing them from the time-consuming operations of writing out work orders, placement of workers, etc.

Technological charts make local operational planning unnecessary. Their use will improve the quality of work and facilitate control over fulfillment of work norms, consumption of materials, and control of prices. It will permit the transition to more advanced methods of labor organization based on the work of teams with complex qualifications.

The table below gives an estimate of savings anticipated from the adoption of the recommended technological charts. The estimate is for each 1000 m^3 of placed concrete.

TABLE

No.	Type of concrete placing	Savings per 1000 m^3 of placed concrete					
		in metal		in equipment,	in labor		total, rubles
		rubles	tons	rubles	man-hours	rubles	
1	Concrete placing from existing trestles with the use of dump trucks	7320	6.65	-	50	520	7840
2	Concreting with the aid of caterpillar cranes (concreting of upstream and downstream aprons)	5150	4.9	1900	60	140	9190
3	Idem (of walls, piers)				90	210	2110
4	Concreting with the aid of tower cranes	-	-	1500	80	220	1720
5	Concreting with the aid of gantry cranes	-	-	900	10	230	1130
6	Concreting with the aid of vibrating trunks	-	-	180	17	390	570

The standard technological charts were examined at the construction sites of the Stalingrad and Kremenchug HEPs and were found to be a valuable tool in the construction practice.

MISI IMENI V. V. KUIBYSHEV CHAIR OF HYDRO CONSTRUCTION

Head: P. V. Borodin, Candidate of Technical Sciences, Lecturer

ANALYSIS OF THE RESULTS OF THE HYDRAULIC FILLING OF THE SARY-YAZIN EARTH DAM ON THE MURGAB RIVER

Scientific Guidance: B. A. Volnin, Candidate of Technical Sciences

Research Team: I. L. Garb, Engineer
Dung Chen-hua, Engineer
V. V. Dudyshkin, Engineer

The purpose of the above investigation was to determine the quality of earth fill of the dam and, if necessary, to prepare recommendations for its improvement.

The quality of the fill was evaluated on the basis of the results of geotechnical investigations carried out during erection of the dam These investigations produced data on the size grade, the bulk weight and the unit density of the earth fill. A total of 1563 laboratory tests were processed.

These investigations showed that the Sary-Yazin dam contains fine-grained powderlike sand of homogeneous size grade (distribution) and of uniform density. The sand has the following characteristics: diameter range of sand grains 0.05 to 0.055 mm, range of maximum grain size 0.10 to 0.12 mm and coefficient of shape nonuniformity 2 to 2.4; bulk weight of earth core is 1.45 to 1.47 ton/m^3. Although the data confirm the good quality of the earth fill, they show that the fill would have been ever better had the filling been carried out by a better technique. Although the design envisaged bilateral filling with the central location of the settling pool, the filling was in fact carried out by a unilateral technique, the pool being alternately shifted toward the faces of the dam. This produced a homogeneous dam core, whereas the design envisaged a heterogeneous core with higher density at the dam shell.

DETERMINATION OF TECHNICAL AND ECONOMICAL INDEXES OF CONCRETE WORK IN HYDRO CONSTRUCTION AND THEIR USE FOR COMPILATION OF ESTIMATES AND PLANS OF CONCRETE PLACING

Scientific Guidance: P. V. Borodin, Candidate of Technical Sciences, Lecturer

Research Team: D. V. Chaplygin, Candidate of Technical Sciences
I. V. Krasnyi, Engineer
L. L. Bocharov, Engineer

As can be seen from its title, the aim of the present work was to establish technical and economical parameters underlying the estimates and schemes for concrete preparation and placing in hydro construction.

The work has been carried out on behalf of the Government Committee for Building and Construction of the Council of Ministers of the U.S.S.R and of the Ministry for Electric Power Plant Construction.

The first phase of this work involved the conduct of time studies at the site of the Stalingrad HEP. They covered: concrete-placing operations at the dam, the powerhouse and the locks; horizontal transportation of concrete mix, reinforcement and formwork as well as haulage of concrete mix through pipelines. These studies covered 62 concreting blocks of the lower tiers of the structures.

In the second phase various data on the performance of concrete work, received from the Stalingradgidrostroi, were collected and processed. Processing permitted compilation of the following guiding material for each of the typical concreting blocks: calendar schedule for all concrete work carried out at the block; reasons for various interruptions in the work of one shift; figures for manpower, equipment and material requirements; technological scheme for the performance of concrete work containing all the relevant technical and economical parameters.

These investigations permitted the reseacher to establish not only the actual and the recommended (improved) duration for each type of operation, both separately and for the whole complex of concreting, but also permitted the calculation of the required manpower, building equipment, and materials. For horizontal haulage of the concrete mix the staff established the actual and recommended duration in cases of both railroad and automotive transport for various types of loads, and calculated the efficiency of horizontal haulage per shift.

For haulage of concrete mix by pipelines, the staff established the actual and the recommended capacities of the various concrete-mix pumps and calculated their efficiency per shift.

The work permitted the staff to draw up plans for better use of manpower and building materials, mechanisms and transportation facilities, thus providing the contractor with basic data on production norms and estimates for concrete work.

The report contains certain remarks and suggestions for better planning of concrete work, e. g. for reducing the time of concrete placing, for improving the procedure of concrete placing directly from dump trucks and from portable platforms, for improving the degree of readiness of the block for subsequent concreting, for reducing concrete-mix losses during haulage, etc.

The report has been handed over to various design institutes of the Ministry for Electric Power Plant Construction.

EXPERIENCE IN RIVER DAMMING AS A BASIS FOR IMPROVING THE PLAN FOR VOLGA RIVER DAMMING AT THE STALINGRAD HEP

Responsible for Research: P. V. Borodin, Candidate of Technical Sciences, Lecturer

Research Team: A. K. Vaidakavichus, Engineer
I. L. Garb, Engineer

The aim of this study has been to improve the design initially prepared by the Gidroproekt for the dam of the Stalingrad HEP on the Volga.

The study makes use of the experience in previous river-damming operations, as well as of the results of field investigations carried out by the Department for Hydro Structures during the erection of dams for the Kakhovka HEP on the Dnieper, for the Gor'kii and Kuibyshev HEPs on the Volga, for the Kairak-Kum HEP on the Syr Dar'ya River, for the Irkutsk HEP on the Angara River, and for the Novosibirsk HEP on the Obi River.

From these data it was possible to determine the magnitude of the final head difference at the dam rock toe for about 30 design alternatives of the tailwater channel. It was found that the magnitude of the final head difference is most strongly affected by the hydraulic losses in the concrete structures and the tailwater channel under conditions of full dismantling of the cofferdams.

From experience in previous river-damming operations it was suggested to carry out the constriction of the river bed with a temporary dike filled from a floating bridge which markedly reduced the volume of subsequent riprap filling. The stability of a dam toe of the new (lightweight) profile was checked by statical calculations.

The study examines four variants of such a sequence of river-damming operations as will permit noninterrupted navigation on the Volga. A new method is presented for hydraulic calculations of the closure of temporary passage ways in the cofferdam with due account of partial accumulations of water in the headrace and its seepage through the rock fill as well as for the graphical determination of the unloading capacity of the floating bridge and hence of the rate of rock filling.

The possibility was proven of closing the temporary passage way (under conditions prevailing at the Stalingradgidrostroi site) at a final head difference of 3 to 4 m. It was shown that, with a suitable rate of rock filling, the hydraulic conditions of river damming will not be inferior to those at the Kuibyshevgidrostroi site. The closure of the passage way in the cofferdam was intended to be carried out with 10-ton concrete tetrahedrons, using riprap or rock fill only in later stages of river damming.

The results of these investigations also proved the possibility of reducing the amount of stone material used in river damming by 150,000 to 200,000 m^3 and the over-all cost, by about 20 to 30 million rubles.

GISI IMENI V. P. CHKALOV DEPARTMENT FOR HYDRO STRUCTURES

Head: E. G. Golikov, Candidate of Technical Sciences, Lecturer

EXPERIENCE IN HYDRAULIC FILLING OF HYDRO STRUCTURES DURING WINTER

Responsible for Research: E. G. Golikov

Research Team: N. P. Kolpashnikov, Candidate of Technical Sciences,
A. L. Suchkin, Engineer

The working of sand quarries (pits) and hydraulic filling of hydro structures during winter somewhat differ from such operations carried out in summer. Thus, the necessity arises of heating the quarry area and the slurry pipelines, of preventing the site of hydraulic filling from freezing and of ensuring proper drainage, of maintaining ice-free sections in the water area around the suction dredges and the floating slurry pipelines.

During the hydraulic filling of nonsubmerged (super) structures, there exists the danger of formation of crystal-ice layers and of soil freezing. In thawing up, such frozen layers may cause increased seepage and even landslides.

Ice-free portions on the water surface are extremely important to winter work, and their proper size and shape should facilitate sufficient movability not only of the suction dredge, but also of the slurry pipeline connected to it. At first sight, there are no particular difficulties in maintaining such ice-free portions, but actually the problem is more complicated.

So far there is no generally accepted practice in conducting such works and the contractors solve these problems each in his own way, the quality of work being often rather poor, which is apt to discredit the merits of such work.

The aim of the present study is to report on the experience in hydraulic filling of hydro structures during the winter of the years 1957 and 1958 with filling material obtained from shallow, closed water bodies.

During this period the following problems were studied: sand quarrying, planning of operation of suction dredges, creation and maintenance of ice-free portions, the use of screw pumps, and general problems pertaining to work during winter.

As a result of field investigations, it was found that hydraulic filling during winter with material obtained from shallow, closed water ponds is absolutely feasible.

Even if sometimes breakdowns of suction dredges occurred, they were not due to difficulties connected with winter work, but to poor quality of the preparatory operations.

Except for the silicate plant and the railway station "Sortirovochnaya" where winter work was merely a continuation of summer operations, the two other sites started working only at the end of the winter season without any proper preparation.

The customer did not put severe demands on the quality of the filling material and did not submit the necessary technical specifications. A radical departure from such practices is required, and new ways must be found to improve planning of work during winter. The reliable performance of hydro mechanization during winter basically requires continuous operation both of work on excavation and of hydraulic filling.

The following conditions make possible continuous operation:

a) the floatability of the suction dredges, which ensures proper cable handling during bottom-lowering dredging (soil excavation);

b) floatability of dredge suection pipes, which ensures reliable conveying of the slurry;

c) faultless state of dredging equipment;

d) continuous power supply to the suctión dredges, ensuring their reliable operation;

e) timely opening of the ice cover and maintenance of the openings on the site where the suction dredge is used;

f) working discipline of the operational staff.

Economical analysis of hydromechanical work carried out during winter shows that the cost of these works increased by 20 to 30% depending upon operating conditions on the site and air-temperature fluctuations.

However, this increase does not exceed the estimated norm for dry soil excavation involving dump-truck haulage over distances of 1 to 2 km; on the contrary, the total is even somewhat lower than the norm establishing the economic utility of hydro-mechanization work during winter.

A comparison of the number of hours of effective work with the number of hours of various outages of suction dredges during the winters of 1955 to 1958 shows that, even in winter a high time coefficient of suction-dredge utilization ($K = 0.52$) is obtainable which is only slightly below the coefficient for summer work.

DEPARTMENT FOR MECHANIZATION OF HYDRO CONSTRUCTION OF THE IGiG OF THE UKR. S. S. R.

Head: N. A. Silin, Candidate of Technical Sciences

INVESTIGATION OF OPERATION OF SUCTION DREDGES AND THEIR CONVEYING PIPELINES

Responsible for Research: N. A. Silin, Candidate of Technical Sciences

Research Team: N. A. Silin, Candidate of Technical Sciences
S. G. Kobernik, Engineer
I. A. Asaulenko, Engineer

The main purpose of the study was to establish optimum operational conditions for large suction dredges. Investigations were carried out during 1954 and 1955, mostly on the 1000-80 suction dredge used at the construction site of the Kakhovka HEP under different field conditions.

The following problems were considered:

1) hydraulic resistance in a 900 mm pressure pipeline during the flow of water and water-soil slurry at a speed ranging from 3.5 to 8 m/sec, and a range of slurry density from 1.05 to 1.20;

2) characteristics of the suction-dredge operation on both water and water-soil slurry.

Apart from these studies on the 1000-80 dredge, certain investigations were also carried out on the 500-60 and 300-40 dredges involving

measurements of delivery, pressure, and pressure losses during the transportation of water and slurry through 800 and 614 mm pipelines.

A venturi tube with a throat diameter of 700 mm was installed on the 900 mm pressure pipeline to measure the flow rate of water and slurry in the pipelines. The specific gravity of the slurry was determined according to the pressure drop between the lower and upper points of the pipeline cross section. The head drop to the pipeline was measured over a 50 m long stretch*.

The full delivery head of the suction dredge was determined from vacuum measurements and the power consumption from the readings of an ammeter and phasemeter installed on the electric motor, with allowance for its efficiency.

The average size grade of sand in the slurry was 0.1 to 0.5 mm.

The tests yielded the following data:

1) hydraulic resistances during the flow of water and water-soil slurry in a 900 mm pipeline;

TABLE

V, m/sec	Coefficient of hydraulic resistance, λ	Head losses on a pipeline stretch of 100 m		
		$\gamma_s = 1.00$	$\gamma_s = 1.10$	$\gamma_s = 1.20$
3.50	0.00966	0.67	-	-
4.00	0.00958	0.87	-	-
4.50	0.00950	1.08	-	-
5.00	0.00942	1.30	1.37	1.45
5.50	0.00934	1.57	1.61	1.65
6.00	0.00925	1.85	1.88	1.92
6.50	0.00915	2.16	2.19	2.22
7.00	0.00908	2.50	2.51	2.55
7.50	0.00899	2.86	2.87	2.90
8.00	0.00892	3.24	-	-

2) technical characteristics of the 1000-80 suction dredge when feeding water and soil-water slurry (Figure 225).

The results of field tests, as shown in the table and in Figure 225, lead to the conclusions that:

a) the coefficient of hydraulic resistance λ depends mainly on the flow velocity even at Re numbers of $(6.0 - 7.0) \times 10^6$;

b) the experimental values of the hydraulic resistances, when compared with the value for new steel pipelines, calculated from F. A. Shevelev's formula, are lower than the latter by 50%;

c) at a speed of 300 rpm and a theoretical delivery head of 80 m, the 1000-80 suction dredge has an output of 14.400 m^3/hr against the 10,000 m^3/hr indicated by the manufacturer;

d) the suction-dredge head on slurry is higher than the head developed on the delivery of water and may be determined from the formula

$$H_s = H_w(0.375\gamma_s + 0.625);$$

* [The table shows a stretch of 100 m].

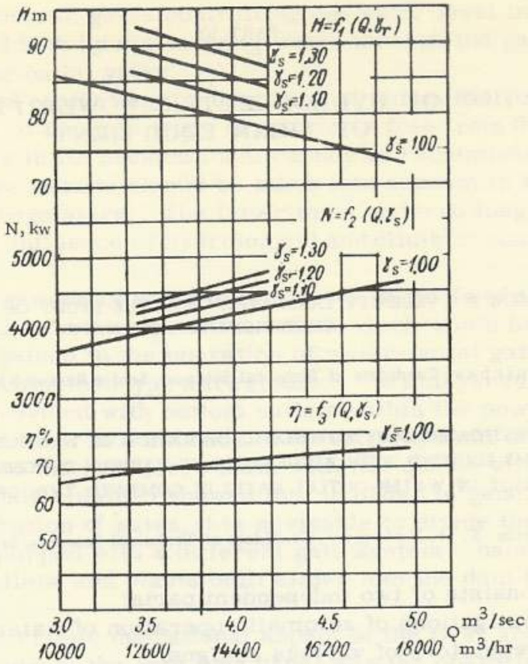


FIGURE 225. Characteristic of the 1000-80 suction dredge operating on water and slurry

e) the actual output capacity (in terms of slurry delivery) of the 1000-80 suction dredge is much higher than its design capacity, because of the increase in the discharge, the rated head remaining unchanged, and because of the smaller head losses in the pipeline, compared with the values usually obtained.

The results were handed over to the All-Union Trust "Gidromekhanizatsiya" of the MES S. S. S. R. and to other interested agencies.

PART VI

OPERATION OF HYDROELECTRIC POWER PLANTS AND OF THEIR EQUIPMENT

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR THE STUDY OF OPERATING CONDITIONS OF HEPS

Head: N. N. Petrunichev, Candidate of Technical Sciences, Senior Research Worker

FIELD INVESTIGATIONS OF AUTOMATIC OPERATION OF WATER-OUTLET GATES IN DAMS EQUIPPED WITH REGULATORS OF VARIOUS DESIGNS, AND THE STUDY OF WATER-OUTLET GATES IN COMBINED TYPE OF HEPS

Responsible for Research: S. M. Nalimov, Candidate of Technical Sciences, Senior Research Worker

The study consists of two independent parts:

a) field investigations of automatic operation of water-outlet gates in dams equipped with regulators of various designs;

b) study of water-outlet gates in combined HEPs;

1. The system of over-all automation of HEPs includes automatic control of water levels in the headwater section of the hydroelectric plant and of the forebay; automatic filling and emptying of the pools for daily regulation; and finally, automatic control of water discharge into the diversion channel in accordance with the water-flow system.

Water levels are controlled by two basically different methods: by automation of the gate drive, or by the use of hydraulically operated fully automatic gates.

It has not yet been determined which automatic-regulator type is the most suitable for given operating conditions.

The aim of this work was the study of four automatic gates of various designs – under field conditions and under identical operating conditions – in order to obtain objective data on their technical characteristics.

A special method was devised for these investigations with the final aim of determining relationships between the position of the gate (i. e. the operation of the water outlet) and the fluctuations, with time, of the water level in the headwater section, that is, the operational characteristics of the automatic gate designed to maintain a definite predetermined water level in the headwater section. Comparison of relationships obtained for various types of gates made it possible to evaluate their operational characteristics under identical operating conditions.

The automatic regulators (gates) were tested at small outlet discharges (~ 20.0 - 25.0 m^3/sec) which limited the fluctuations of the outlet discharge. The basic operational characteristics of the automatic gates were recorded by means of a limnigraph and a chronograph. The limnigraph recorded water-level fluctuations, while the chronograph recorded changes in the working position of the gates. Both recorders were synchronized, thus ensuring that, in terms of time, their indications would be fully comparable.

Operation of automatic gates controlling the water level in the headwater section was checked both by discharging water through the gate and by the overflow through the outlet valve.

By comparing and evaluating the operation of automatic control gates, it is evident that none of the gate types examined is free from the phenomenon of overshoot, and hence in its present form cannot be recommended for wide use in HEPs. These results should be taken into account in the future designs of automatic regulators. The latter must undergo long-time tests in order to clarify the influence of hydrological and climatic conditions on their operation.

2. Field investigations of water-outlet gates in HEPs of the combined type were conducted to obtain operational data which would help to generalize the experience gained in the operation of water-outlet gates.

Five HEPs of the combined type were studied: the Kuibyshev, Kakhovka, and Dubossary plants, provided with bottom outlets within the powerhouse: the Kama plant which is built into the body of the spillway dam; and the Ortachal'skaya plant, built into the dam piers (dam with flat double sliding gates).

Analysis of the data obtained showed that in order to generalize the experience in the operation of gates, it is advisable to divide the plants into two groups, each equipped with a different gate system - namely, plants combined with water outlets, and plants built either into the dam body or into the dam piers.

The most characteristic operational aspect of the first group of plants (from the point of view of the gate type used) was the operation of working gates at the bottom outlets located, as a rule, at the downstream part of the outlets. Operation of the other gates (their type and construction) used at these plants, does not differ basically from the operation of gates at conventional hydro plants.

The most characteristic features in the operation of mechanical equipment of the second group of plants (built into the body of dams or of piers) were the operation of the spillway gates (at the Kama HEP), and of the quick-dropping gates mounted in front of the turbines (at the Ortachal'skaya HEP).

Operation of bottom gates at a relatively high water head in water outlets combined with the powerhouse revealed:

a) the extremely unsatisfactory state of packings;

b) the uneven character of operation caused by gate vibrations which occurred during water discharge through partially opened outlets;

c) the need to exert additional pressure on the gates when they set onto the sill.

These shortcomings are a serious obstacle in the normal operation of hydro plants, because they demand the special attention of the plant's technical personnel.

Analysis of the causes of shortcomings in bottom water outlets and of failures occurring during their operation, and a study of pertinent foreign technical information indicate that neither in the planning of the gate layout nor in the design of the gates and gate elements, were the findings and the operational experience of other countries with identical gate designs taken into account.

Specifically, foreign experience points the way to improvements, such as:

a) greater efficiency in the operation of packings through more suitable location of the gates at the upstream section of the water inlet channel and

through the use of the hydraulic type of packings; the latter permits the introduction of a vital principle in proper packing operation – the possibility of breaking contact with the packing at the moment of displacement of the gate;

b) elimination of the need to press the gates forcefully at the moment of their closure (setting); elimination of vibrations in the gate by giving its lower section a special shape with its convexity toward the headwater and by providing for aeration of the space behind the gate;

c) eliminating the need to subject the bottom water inlets to the full head of the headwater body;

d) improvements in the method of emptying draft tubes with the aid of stoplog gates, at the same time eliminating the possibility of flooding the downstream screenhouse by the headwater.

Comparison of results of analysis of gate operations in bottom water outlets at combined HEPs shows that the most suitable location for working gates is in the upstream section.

Such location of gates will in turn predetermine the relative layout of the openings in height: the water inlet will be located higher than the outlet.

Therefore, from the point of view of efficient use of bottom outlets, the Dubossary HEP with its spillway inlets located high and its operating gate located at the upstream side of the spillway should be considered as the most efficient type of a combined hydro plant.

Difficulties in the operation of gates at spillways of the built-in power plants (Kama HEP) derive from poor design of equipment, specifically of the gate sealing, and of the system for coupling the gates by means of cranes. Difficulties also arise during the operation of gates located in open-air screenhouses, particularly in the autumn and winter seasons.

The operation of gates in the water outlets of pier-type powerhouses has certain advantages over that of gates located in the water outlets of the built-in type of power plants. These advantages are due mainly to the fact that the functioning of gates in pier-type plants does not differ basically from the functioning of gates in conventional power plants.

Thus, even a brief experience in the operation of gates used at combined hydro power plants shows that, from the point of view of efficient functioning of mechanical equipment, the most suitable type of hydro plant is the one whose bottom spillway is designed to place the water inlet higher than the water outlet, the working gates being installed at the upstream side of the structure. Such an arrangement envisages a gate design based on the latest field and laboratory findings.

THE STUDY OF EXPERIENCE WITH OPERATION OF HYDROELECTRIC PLANTS, WITH THE VIEW OF IMPROVING EFFICIENCY AND OPERATING CONDITIONS OF THE POWER SYSTEM

Research Team: O. V. Vyazemskii, Candidate of Technical Sciences, Senior Research Worker
N. N. Yagodin, Senior Research Worker
S. M. Nalimov, Candidate of Technical Sciences, Senior Research Worker

Within the framework of the research plan of the VNIIG, the Institute gathers data which clarify the operational shortcomings of HEPs, analyzes their causes, and submits recommendations relevant to the proper design and operation of HEPs.

During 1958 the Institute investigated the Perm HEP located on the Kama River, and the Upper-Svir'HEP. The investigations covered the whole complex of problems affecting power generation (various types of power losses), organization convenience, reliability, and operating cost. Plant designs, the quality of structural work and its effect on plant operation were scrutinized. Operation of mechanical equipment was studied along the same lines.

To ensure the widest scope of the investigations, each of the HEPs was investigated by three groups of research workers who studied: a) the hydrotechnical problems; b) power losses in the turbine units; c) mechanical equipment.

Investigations at the Perm HEP showed that the operational shortcomings relating to the hydrotechnical aspect fall into two groups:

a) independent of the given plant type;

b) dependent on the given plant type.

Among the defects of the first group should be mentioned:

1) absence of a protected forebay at the gate approaches, although such a forebay is required by the present Technical Specifications governing the design of navigational structures in hydro power development;

2) poor layout and arrangement of superstructures - specifically, the absence of a separate control house - even though the superstructures were partially redesigned by the operational staff of the plant;

3) lack of facilities for bringing the lock gates to the repair shop; this necessitates repair work outdoor;

4) poor design of the dewatering system of water intakes of the spiral casings and the draft tubes during repairs;

5) lack of provision for cooling the air in the submerged structures of the HEP by river water, resulting in excessive heat inside these structures during the summer.

Actually these shortcomings are not detrimental to the type of the spillway HEP as such; but they can be completely eliminated by a more efficient design of the power plant.

More important are the faults inherent in the type of the so-called "spillway HEP". Among them should be mentioned:

1) an excessively large number of small-capacity power units, causing increased power losses and difficulties in power-plant operation; the Perm HEP could have been operated with 6 larger turbine units and 8 larger gates, instead of the 24 smaller-capacity turbine units and the 24 smaller gates;

2) increased head losses amounting to 48.788 thousand kwhr/annum, which cannot be compensated for by using the ejection effect which would provide additional 36,500 thousand kwhr/annum; there are also losses due to the clogging of trash racks amounting to 55,838 thousand kwhr/year. The total annual loss of more than 26.4 million kwhr make this type of power plant unprofitable.

3) labor-consuming character of operations and adverse physical working conditions.

Investigations carried out at the Perm HEP to find an efficient way of eliminating clogging of trash racks by floating timber or other objects disclosed the following:

1. Serious flaws in the design of the Perm HEP are a source of difficulties in its operation. These faults result mainly from a failure to anticipate the extent to which trash racks are clogged by floating timber. The design of spillway-type HEPs should provide special measures to reduce power-output losses, and improve plant operation.

2. In designing power plants of the combined type on rivers carrying large amounts of debris, preference should be given to HEPs with bottom water outlets, rather than to the spillway-type of plants. In this case the design should provide a special layout of the trash racks, i.e. locating them outside the restricted site, an arrangement that would reduce head losses at the racks, and facilitate the operation of rack-cleaning equipment.

3. Difficulties occurring during cleaning of trash-retaining racks and gates are due mainly to the poor design of the cleaning equipment.

4. Apart from the efficient and necessary means for preventing the clogging of trash racks by floating debris used by the management of the Perm HEP, new ways should be found of improving methods of floating wood and rafting, or of quickly clearing the forebay of floating debris.

Investigations made at the Perm HEP point to the following conclusions:

1. One of the major obstacles to normal operation of mechanical equipment is the strong seepage through the spillway gates, due to faulty design of the water seals. Water seepage through the gates leads to two types of losses:

a) power-output losses due to seepage of water reaching a flow rate of 15 m^3/sec;

b) loss of money spent for de-icing the gates frozen at their tailwater side.

2. Another obstacle to normal plant operation is the need to operate the turbines without protection against runaway caused by breakdown of the hydraulic hoist of the quick-dropping gates.

The defects mentioned above are due mainly to the low level of engineering, and partly to poor utilization of equipment.

Unsuitable design of mechanical equipment, determined by the type of spillway hydro plant as such, is liable to cause a series of other difficulties. Such shortcomings of the design are:

1) absence of a guard plate above the screen wall of the powerhouse;

2) location of the gates in a common recess with the stoplogs.

In view of the large accumulation of floating debris in front of the gates, separate placement of the gates in front of the stoplogs is essential; unfortunately, the spillway type of HEPs precludes the possibility of such design;

3) use of plane (flat) sliding gates for closing the water outlets, thus precluding the possibility of topping floating objects over the crest of the gate and causing a dangerous increase of specific water discharge in the plant's tailwater channel.

It would be more practical to use hinged-leaf gates of V. G. Gebel's design, made of tubular beams of lens-shaped cross section, since these are more suitable for dams having a smaller number of large-size spans (bays). This design would simplify operation of equipment and raise the efficiency of its utilization.

The study of operating conditions of the hydro-engineering structures at the Upper-Svir' HEP was conducted with the view of analyzing (a) the routine operation requirements and (b) general problems of design.

From the point of view of routine requirements, operation of this plant raises no special problems except for the following shortcomings:

a) impossibility of dewatering the drainage galleries;

b) the handling of the cumbersome, rarely operating segment gates of the dam.

The hydro development, from the point of view of the equipment, is ready for normal operation.

The operational effectiveness of the Upper-Svir' HEP could have been enhanced by a better handling of the backwater-level problem at Lake Onega, and by the location of the plant site farther downstream. Such a solution would have reduced the volume of hydraulic-cleaning work in the upper and lower reaches [of the plant]. Moreover, it would have reduced the over-all cost of the hydro development considerably if the conventional, separate spillway dams were replaced by bottom water outlets located below the mounting bay.

Such a design, while incorporating the basic advantages of a combined HEP – as the absence of a separate spillway dam – is also free of its basic shortcomings which are: increased head losses in the turbine unit as a result of smaller over-all size of the turbine intake and the spiral casing.

The investigations at the Upper-Svir' HEP lead to the following conclusions:

1. Normal operation of the plant is hindered by serious difficulties in the operation of mechanical equipment.

2. The most important operational defect of the gates is their susceptibility to seepage, owing to faulty design of their water seals (in Taintor gates) and to poor installation (of quick-dropping gates).

3. Normal operation is greatly hindered by defective layout of the screen house equipment, resulting in unjustified expenditure of money.

4. Poor operational servicing of mechanical equipment has caused breakdowns in the operation of the segment gates, of air-blowing equipment, etc.

5. Since valve gates made of tubular, beams of lens-shaped cross section seem to be better suited for the operation of the water outlets of the Upper-Svir' dam than segment gates, the question of whether to use the latter at the Upper-Svir' dam can be answered only by economic comparison.

SCIENTIFIC AND ENGINEERING STUDIES OF DESIGN OF REMOTE-CONTROL HYDRAULIC MEASUREMENT EQUIPMENT FOR THE GOR'KII HEP

Responsible for Research: I. A. Chernyatin, Candidate of Technical Sciences, Senior Research Worker

Between 1952 and 1954 the VNIIG provided systematic technical advice to the design department of the Mosgidep on the design of equipment for control of river discharge and of turbine operation at the Gor'kii HEP. It also helped to place orders for suitable equipment. This involved frequent personal contact of VNIIG representatives with Mosgidep in Moscow or at the offices of VNIIG.

In 1957 representatives of the VNIIG visited the Gor'kii HEP for a check-up of the progress in the installation of the control and measuring equipment and of preparations for its rating and calibration.

In 1958 the VNIIG:

a) completed the preparation of this equipment for rating;

b) carried out the rating of the level gages of the headwater and tail-water areas of the turbine flow-rate meters, and of the 8 sets of head-drop meters at the screens and gates;

c) conducted efficiency tests on eight turbine units for low and high water head;

d) carried out the rating of the recording and the integrating elements of the turbine flow-rate meters;

e) investigated head losses in the water-conveying galleries of the turbine unit No. 1- first with installed, and then with dismantled trash-retaining screens.

Tests of efficiency of turbine units were carried out by a simplified method whereby, instead of measuring the turbine discharge directly by a conventional current meter (a time-consuming and difficult operation), the head drop in the spiral casing, interrelated with the water discharge, was measured.

Despite the identical design of the turbine units and of the spiral casings, the optimum zones of various turbine units, which correspond to maximum turbine efficiency, were different. The turbine units also reacted differently to the reduction of head.

The efficiency of each turbine unit can be assessed in detail by comparing the performance of each turbine unit designed by the Institute.

Investigations of head losses in the waterway from the headwater level to the entrance into the spiral casing— first with installed, and then with dismantled trash racks - showed that in addition to losses on the racks great losses also occur in their recesses.

Results of field studies showed that for a water discharge through the turbine unit (Q = 500 m^3/sec), the losses of head (not the head drop) at the trash racks amounted to about 17 cm water gage whereas the total head losses on the way from the headwater up to the spiral casing amounted to 36 cm water gage. The head losses in the recesses amounted to a mere 14 cm water gage.

Head losses on the way from the headwater up to the entrance to the spiral casing – with the same discharge rate of 500 m^3/sec, but with dismantled trash racks – amounted to 28 cm w. g. of which 23 cm drop at three rows of rack recesses. These field investigations, the first ever carried out in the U. S. S. R., demonstrated the need to take into account head losses deriving from the design of gate recesses. It can be assumed, roughly, that one row of recesses causes a head loss of about 4 to 5 cm w. g. at a water-flow velocity of about 1.6 m/sec.

It should be noted that the presence of three piezometric gages in the water-conveying galleries of the turbine – the first in front of the racks, the second behind them, and the third in front of the entrance to the spiral casing – greatly facilitates the conduct of such investigations, and permits separate computations of head losses on the trash racks, in recesses, and on the waterway walls.

According to information from the management of the Gor'kii HEP, the operation department of this plant decided to replace the existing racks.

In the light of the above results, the use of new racks, in the author's opinion, will not lead to any essential reduction in the head losses. This is also confirmed by the fact that for the most frequently encountered turbine discharge Q = 350 m^3/sec [any type of] turbine racks causes head losses of ~ 8 cm w. g.

FIELD INVESTIGATIONS OF HYDRAULIC TURBINE UNITS, AND NEW TYPES OF FLOW-RATE METERS INSTALLED AT ONE PLANT OF THE SVIR' CASCADE SYSTEM

Responsible for Research: I. A. Chernyatin, Candidate of Technical Sciences, Senior Research Worker

In 1958 the VNIIG tested the hydraulic measurement devices and the turbine units of the Svir' HEP No. 12 and investigated the new flow-rate meters used at this plant.

Testing of Hydraulic Measurement Apparatus

1. In order to check air accumulation in the piezometric tubes of the hydraulic flow-rate meters, the VNIIG designed and manufactured eight transparent air chambers. These were mounted on all flow-rate meters of HEP turbines. After the first 25 days of operation no noticeable air accumulation in the air chambers was observed. For the first seven and a half months of continuous operation at the positive-pressure branch of the flow-rate meter of one turbine, about 20 cm^3 of air were found in the air chamber. In the other air chambers, mounted on both the positive and the negative pressure branches of the flow-rate meters, an accumulation of about 2 to 3 cm^3 of air was observed.

Operational experience confirmed the advisability of placing the transparent air chambers at the highest points of the piezometric tubes. They are easy to handle and do not require the air to be removed nor the tubes to be flushed, except in special cases.

2. The flow-rate meters installed at the turbine units were carefully tested for: head-drop recording, simultaneous response of the integrating elements and the recording pointer, constancy of counting-mechanism operation, and finally, for proper functioning (by proportioning the amount of mercury in the device) of pointer zeroing and of the lever mechanisms.

3. The turbine flow-rate meters were rated in mounted condition, the scale being later calibrated in direct discharge-rate units.

4. The piezometric tubing of the flow-rate meters was tested for air-tightness. The principal tube of the system, laid out from the external wall of the spiral casing, was found to be clogged.

Operational Tests of the Turbine Units

In order to establish the actual performance of the HEP turbine units, three of them were examined on the spot by a simplified method. Tests were carried out at both high and low water head.

These investigations made it possible to ascertain the optimum working conditions at which turbine units operate with maximum efficiency. They also showed that a suitable distribution of load among the turbine units can be made by separate additional computations for each value of head.

Investigation of New Types of Flow-rate Meters

1. To ascertain which type of flow-rate meter has the most accurate counting mechanism for measuring the flow rate through the turbines, the Lenenergo Administration installed two new types of flow-rate meters at the Svir' power plant No. 12.

Under the guidance of the VNIIG staff, and following its blueprints, the power-plant staff installed the meters on one of the turbines.

2. Preliminary tests were made in December 1958 of the counting mechanisms of the two new types of meters and of the previously installed flow-rate meters.

TESTING AND IMPROVEMENT OF OPERATION OF HYDRAULIC MEASUREMENT DEVICES (Flow-rate Meter and Head-drop Meters) USED IN HEPs

Responsible for Research: I. A. Chernyatin, Candidate of Technical Sciences, Senior Research Worker

On the above subject the following work was carried out by the Institute in 1958.

a) tests of hydraulic devices and piezometric tubes under field conditions;

b) observation of air accumulation in pressure-transmitting pipelines from the pressure transmitters to the recording instruments;

c) research on subject matter, using manufacturers' catalogs and other available information;

d) design of an improved turbine flow-rate meter.

With regard to (a), the tests of device operation were carried out at the Irkutsk, Novosibirsk, Gor'kii and Upper Svir' HEPs. As far as the float system is concerned, it was found necessary to heat the float itself, rather than the shaft where the float is mounted. For this purpose the Institute designed an electrically heated float. The terminals of the piezometric tubes (lines) through the concrete layer should be located at least 200 mm from the vertical walls.

The detrimental effect of load conditions in adjacent turbine units on the magnitude of head drop at the gates was observed in field investigations. To obviate such effect in future designs, gages for measuring the head drop at the gates and separate pressure pick-ups should be installed in front of and behind the screens of each turbine unit.

With regard to (b), field operation of transparent air chambers of flow-rate meters, mounted at the Upper-Svir' and Gor'kii HEPs, showed small accumulations of air in the upper area of the pipelines – from 2 to 25 cm^3 – after almost a year of operation. This study should be continued at a number of other HEPs where transparent air chambers should be installed for this purpose.

With regard to (c), contact was made with instrument-manufacturing plants and their production programs studied. The study of new types of instruments and their utilization for hydro power plants will be continued.

With regard to (d), field investigation of turbine flow-rate meters carried out at a number of HEPs revealed shortcomings in the operation of both the recording and integrating elements. Research was continued on improvement of the design of flow-rate meters, which would ensure correct integration of discharge recordings by the counting mechanism.

This research work has been carried out in two directions: 1) by operational check-up of flow-rate meters, and 2) by comparison of the design and operation of various types of operating flow-rate meters.

VNIIG IMENI B. E. VEDENEEV LABORATORY OF HYDRO TURBINE UNITS

Head: A. M. Chistyakov, Candidate of Technical Sciences, Senior Research Worker

STUDY OF SILT FORMATION AT HYDRO POWER PLANTS SUBJECTED TO INTENSE MECHANICAL EROSION OF TURBINES UNDER THE ACTION OF SILT

Responsible for Research: V. B. Dul'nev, Candidate of Technical Sciences, Senior Research Worker

Many operating HEPs located on mountain rivers experience great difficulties from the abrasive wear of turbines caused by silt erosion. Experiments were carried out by the LMZ on operating turbines of a number of HEPs, with the view of making turbine parts more wear-resistant. The turbine blades of the Baksan, the Verkhnii-Varzob, and of the Rion HEPs, were reinforced by built-up welded layers, while experimental protecting rings, made of various materials, were installed on the lower pivots of the guide blades. The reinforced parts were tested between 1955 and 1958.

For the correct evaluation of the test findings it is necessary to know the characteristics of silt formation in water passing through the turbines during the test period. Consequently, the present study dealt with silt formation at the above three power plants during the test period covering the years of 1955, 1956 and 1957.

During these years, the plant personnel took daily samples of silt-laden water at the outlet of the turbine draft tubes. These samples were first processed on the spot – by settling, evaporation, and weighing – and then sent to the VNIIG for mechanical and mineralogical laboratory analysis. The data acquired by observations (see table below) and by laboratory analysis, supplemented by operational data (number of hours of operation, water discharge through turbines, etc.) led to conclusions regarding silt deposition in the turbine units concerned.

For the test period, silt formation is characterized by the average annual concentration of all silt particles P, and of erosive solid (mineral) particles P_{min}.

TABLE

Basic data on silt deposition in the turbine units of the Rion*, Baksan, and Verkhnii-Varzob HEPs for 1955, 1956 and 1957

Name of HEP	Head H, in m	Turbine type	Turbine wheel diameter, m	1955			1956			1957		
				Average hours of turbine operation	Amount of total silting particles in water, P, g/liter	Amount of erosive mineral particles in water, P_{min}, g/liter	Average hours of turbine operation	Amount of total silting particles in water P, g/liter	Amount of erosive mineral particles in water, P_{min}, g/liter	Average hours of turbine operation	Amount of total silting particles in water, P, g/liter	Amount of erosive mineral particles in water, P_{min}, g/liter
1. Baksan	90	PO-82	1.2	5300	0.86	0.37	6200	0.79	0.33	6200	0.85	0.36
2. Verkhnii-Varzob	47	PO-80	1.2	6950	0.21	0.06	7250	0.12	0.03	8100	0.22	0.07

* [Though indicated in the table caption, no figures for the RION HEP are given]

The results of the investigations have been passed on to the LMZ plant. They may be useful for the correct evaluation of results of tests on the wear resistance of turbine components, and for the proper selection of wear-resistant materials, needed both for the manufacture and repair of the water flow-path elements of turbines operating at mountain-river HEPs.

VNIIG IMENI B. E. VEDENEEV ICE-THERMAL LABORATORY

Head: Professor A. M. Estifeev

STUDY OF HYDROLOGICAL PROBLEMS RELATED TO THE OPERATION OF SOME OF FINLAND'S HYDRO PLANTS IN WINTER

Responsible for Research: Professor A. M. Estifeev

This work availed itself of the preliminary report on ice and thermal conditions prevailing in the rivers and in other bodies of water of Finland, prepared by the author after a sojourn in Finland, where he visited both the operating plant, and plants in construction on the Oulu and Kemi rivers.

The author also visited and studied the work of the Hydrographic Administration in Helsinki, the Main Administration of the Imatran Voima Company dealing with research work, design, and operation of the principal HEPs in Finland.

The following conclusions were drawn from this preliminary study:

1. Generally, the methods used in the U.S.S.R. for studying ice and thermal conditions of rivers are of a higher standard than those used in Finland.

2. A positive feature in the field of hydrological study of rivers and of other Finnish water reservoirs is the dense network of hydrometric stations, which permits an exact appraisal of water resources and timely forecasting of river runoff.

3. Other positive features include:

a) the air blow-off system which prevents the freezing of the river near the dam gates;

b) wide-scale use of frazil-ice-retaining booms;

c) the use of special guns "Kandapolit" for crushing the ice cover; the shock waves produced spread horizontally, and are thus instrumental in the loosening and removal of frazil-ice accumulations;

d) the use of the cascade system of back-water storage lakes; such a continuous chain of lakes markedly improves operational conditions of the power plants during the winter;

e) increase of river depth by bottom excavations and the construction of special diversion channels at particularly low river-bed points, to prevent flooding which may occur as a result of the creation of storage lakes.

This study, in addition to describing the principal power plants visited, includes blueprints and photographs.

PLANNING OF OPERATION OF HYDRO STRUCTURES AND OF HYDRAULIC MEASUREMENT EQUIPMENT AT THE NARVA HEP

Responsible for Research: S. M. Aleinikov, Junior Research Worker

The purpose of this work was to find ways of improving the utilization of hydro structures in winter, and to compare the results of field investigations with the appropriate design data.

During the autumn-winter periods of 1955-1956 and 1956-1957 field studies were made of ice and thermal conditions of the storage lake and the diversion channel.

The winter of 1955-1956 was particularly cold. At the channel entrance the water temperature was as low as + 0.025° C on 3 February, and by 13 February had dropped to + 0.01° C. Along the channel the temperature gradient was between 0.02 and 0.03° C; subcooling of water and formation of submerged ice was observed.

Comparison of measured water temperatures with the rated values gave satisfactory agreement.

Ice accumulation was observed in the forebay and the ice cover extended along the channel up to the bridge.

The nature and size of unfrozen patches in the ice cover of the storage lake confirmed the predicted values.

During the winter season of 1956-1957 the power plant operated under much more favorable conditions. The temperature gradient along the channel did not exceed 0.01° C; the zero isotherm shifted downstream to the tailwater of the HEP, as foreseen by the plan.

Comparison of the two winter seasons (1955/1956 and 1956/1957) showed that during the steadily cold winter of 1955/1956 the high prevailing flow velocities in channel created considerable operational difficulties for the plant. On the other hand, when freezing and warm periods alternated, high flow velocities in the channel entailed lesser difficulties. Moreover, during low-temperature periods, high flow velocities call for special measures for clearing the channel of ice, which had not been done during the winter of 1955/1956.

The ice-removing structures were finished only in 1956-1957; they effectively cleared the 1.5 to 2 cm layer of ice formed in the forebay of the HEPs.

The chief conclusions of the present study are:

1. It was confirmed that when using the existing water-conveying structures for protection of the plant against ice troubles, it is desirable to start the sequence in using these structures from the upper reaches of the river. Ice pressure observed at the Omutskie rapids (in the region of backwater fading at the headwater section of the Narva HEP) is a cause of serious trouble in the operation of this HEP. Construction of the Omutskaya HEP will eliminate these troubles.

2. Experience gained from the plant operation during the period from 1955 to 1957 shows that normal plant operation in winter can be ensured provided appropriate measures are taken. The ice-removing structures can function even without the installation of ice-retaining booms in the forebay.

3. A special computation method has been suggested for determining the resistance of canal revetments (linings) to shore ice.

INVESTIGATION OF ICE AND THERMAL CONDITIONS AT THE HEADWATER AND TAILWATER SECTIONS OF THE GOR'KII HEP

Responsible for Research: I. N. Sokolov, Candidate of Technical Sciences, Senior Research Worker

Apart from the study of ice and thermal conditions, the purpose of this work has been to elaborate recommendations for increasing the efficiency of the Gor'kii HEP and of its power output in winter.

In 1958 the laboratory made the following studies connected with this work:

1) analysis of data on winter conditions at the upper and lower reaches of the HEP (data were obtained from the Hydrometeorological Bureau, Mosgidep, Orgenergostroi, and the Main Administration of the Gork'ii HEP);

2) field investigations: a) on the effect of wind-driven waves in cooling the headwater, and on the subsequent formation of ice; b) of thermal conditions of the water in both reaches, particularly in front of the spillway gates; c) on the effect of formation of ice cover and frazil ice, in the storage lake, on the performance of the HEP;

3) calculation of static pressure of ice on the sliding gates.

Information was also obtained and analyzed on the question of setting up air-blowing units to maintain an unfrozen portion in the ice cover in front of the gates. Experience gained in the operation of such units, mounted at

the Svir' HEPs was studied for application in the appropriate adjustment of the air-blowing unit at the Gor'kii HEP.

From this work the following conclusions were drawn:

1. It was established that during the past years there was no formation of frazil ice either in the headwater area, or at the plant itself.

2. Formation of ice cover on the storage lake occurs very rapidly and over a great area; in the powerhouse area there were no unfrozen patches in the ice cover.

3. The temperature of the water upstream from the plant in front of the powerhouse and at the dam gates, prior to freezing, is almost uniform throughout its depth, and is close to zero. During the first period after the formation of the ice cover the temperature remains at the same level.

4. The water temperature in the tailwater section is close to the average (related to depth) temperature in the headwater section. This can be explained by the more or less uniform intake of water from the upper and lower stream layers (the graph of water-flow velocity distribution below the ice cover assumes the shape of a smooth parabolic curve).

5. During ice-cover formation, wind-driven waves on the storage lake and near the plant may attain considerable proportions: in the area near the plant the observed height of the wind-driven waves reached 1 m. In 1958, however, despite such high waves, the storage lake froze in the course of a single night.

6. On the Volga, downstream from the Gor'kii plant, ice-cover formation develops much more slowly. The propagation velocity of the ice-cover edge downstream depends basically on meteorological conditions.

On completion of this work recommendations were proposed for the improvement of winter operation of the plant.

TNISGEI IMENI A. V. VINTER HYDRAULIC LABORATORY

Head: L. G. Gvelesiani, Senior Research Worker, Lecturer

AN INDEX OF POWER-ENGINEERING EFFICIENCY OF DAILY REGULATION, AND OPTIMUM ADAPTATION OF HEPs NOS I AND II TO THE "n" STEP 24-HOUR LOAD CURVE

Responsible for Research: N. A. Tsabadzhe, Junior Research Worker

Proper distribution of the 24-hour load curve of a power system among the separate power plants is particularly difficult for a "mixed" (combined type) of power system comprising both hydro and thermal power plants.

The losses in hydraulic, heat, mechanical and electric power connected with daily regulation vary in dependence on the methods for using available power sources, the type of power plant and the amount of average daily water (Q_b m^3/sec) or fuel (q_b kcal/hr) consumption for power generation, and do not lend themselves to comparison by absolute values. However, in solving the problem of adapting a hydro power plant to the 24-hour load curve power system, we must have a criterion showing which power plant, possessing a

certain degree of daily regulation, has to take care of the peak, semi-peak or the base load of the curve.

The suggestion is made to use, as such a criterion, the concept of the "factor of power-engineering efficiency of daily regulation" ($\Theta_{d.r.}$) expressing the ratio of peak load (E_p) to the base load (E_b)

$$\Theta_{d.r.} = \frac{E_p}{E_b} \leqslant 1. \tag{1}$$

where $E_b = 24 \times N_b$ kwhr (24 hours of a day), where N_b is the basic capacity in kwhr corresponding to a water discharge ($Q_b \, m^3/sec$) or fuel consumption (q_b kcal/hr) of the thermal power plant;

$E_p = t_p \times N_p$ kwhr, where N_p is the full peak capacity in kw (i. e. maximum working capacity at full load of all generating sets); t_p is the number of hours of power-plant operation at peak load at full transformation of Q_b (or q_b) into Q_p (or q_p) corresponding to N_p (maintaining the balance of water or fuel consumption, over 24 hours).

For a HEP t_p is determined from the condition of equality between discharge and filling of the storage lake

$$3600 \int_0^{t_p} Q_p \, dt = 3600 \, Q_b (24 - t_p), \tag{2}$$

when 3,600 = number of seconds/hour.

Equation (2) is rather difficult for practical use, taking into account the complex functional dependence of Q_p on the varying head at the HEP during peak-load operation t_p. Therefore, the problem should be solved by the iterative method.

For a thermal power plant we obtain

$$t_p = 24 \frac{q_b - q_0}{q_p - q_0}, \tag{3}$$

where q_0 in kcal/hr is the minimum fuel consumption for maintaining given steam conditions when the plant operates on reserve only.

Curves plotted from (1) in (P, $\Theta_{d.r.}$) coordinates for various values of $P = \frac{Q_b}{Q_p}$ $\left(\text{or } P = \frac{q_b}{q_p}\right)$ permit the comparison of various types of power plants in solving the problem with regard to their qualitative aspects, i. e. the proper adaptation of electric power plants to the daily load curve (forecast 24 hours in advance) of the power system, with given Q_b or q_b.

The factor (1), as suggested above, fully reflects the process of daily regulation and its consequences (power losses) observed in the electric plants of a power system; the higher the $\Theta_{d.r.}$ with given P, the easier it will be for an electric power plant to take care of daily load fluctuations, with minimum power losses.

The adaptation of a power plant to the load curve of a power system has to be made by starting with the peak loads and proceeding to the base load as the value of $\Theta_{d.r.}$ decreases. The value of $\Theta_{d.r.}$ calculated from (1) can easily

be adjusted with due regard to the power losses in the distribution lines and the transformer.

As a result of investigations, the paper presents curves for the function $\Theta_{d.r.} = f(P)$ characterizing the operation of power plants of the Georgian power system These curves, plotted by analyzing available design and operational data, may be useful to the load-distribution (dispatcher) department of the power system.

To determine the economic effect from the adaptation of the experience at the Georgian power system it is necessary to carry out additional power-engineering and economic calculations, i. e. comparison of actual operational data of plants with their adaptation to the load curve, according to condition $\Theta_{d.r.} = f(P)$.

The paper also presents a combined graphical and analytical solution to a concrete case of operation: the optimum distribution for high-head pondage HEPs I and II in the "n-stages" scheduled load curve for the Georgian power system (with known duration of each stage) at given daily average water discharges (forecast 24 hours in advance) into the storage lakes of the above plants; daily regulation is understood here to be carried out without disturbing the water balance throughout the whole day (i. e. 24 hours). This graphical-analytical method also takes into consideration the power losses in the distribution line and transformers of the above plants, from the place of connection of plant I and II to the general network of the system.

Solution of this problem permits the determination of the optimum variant for daily regulation at HEPs I and II, ensuring maximum over-all power output at both plants; for both plants the shape of the over-all load curve corresponds to the scheduled "n-stages" load-curve of the power system.

The methods described in this paper for calculating daily regulation without disturbing the 24-hour water balance are actually a general solution of the problems of a power system with unlimited possibilities for daily regulation, i. e. when the volume of the HEP water body to be regulated is $W \geqslant 21{,}600\, Q_p\ m^3$.

Similar to work done in previous years, the introduction of the graphical-analytical method at the HEPs I and II alone will afford yearly savings of at least 300,000 rubles.

THE IEVKh LABORATORY FOR HYDRO CONSTRUCTION

Head: K. F. Artamonov, Candidate of Technical Sciences

EXPERIENCE IN THE OPERATION OF WATER INTAKES WITH BOTTOM SCREENS USED IN THE RIVERS OF THE Kirghiz S.S.R.

Research Team: M. S. Ramazan, Candidate of Technical Sciences
V. F. Talmaza, Junior Research Worker

The staff, in cooperation with the Kirgizgiprovodkhoz Design Institute made laboratory and field studies of water-intake structures on small mountain rivers of Kirghizia, in order to eliminate shortcomings in the design, and in the determination of conditions favorable for the use of water intakes having bottom screens.

The field examinations made on five structures involved the study of changes in river channels; the nature of distribution of linear discharges and bottom load along the intake structures; the silting of bottom screens; the silt-carrying capacity of the stream, and the operational conditions of the filtering, settling and discharge channels.

All necessary measurements were carried out by conventional methods, i. e. by determining the flow velocity and discharge of water, the direction of the flow filaments, changes in the river channel, etc.

The discharge of bottom load was measured by screen-type traps (30× ×30×60 cm with mesh opening of 4 to 7 mm), installed tightly on the bottom of the structure apron.

The water-intake structures that were investigated are located at the end of the mountain rivers and at their headwaters, characterized by a maximum water-discharge range of 18 to 70 m^3/sec, an average yearly discharge of 3 to 12 m^3/sec, and a winter runoff of 0.75 to 2.0 m^3/sec. In the area of the water-intake structures with an average slope of 0.027 to 0.059, the river beds consist of pebbles and boulders having a diameter of up to 900 mm with an average thickness of deposits of 150-300 mm.

Five models of bottom-screem water intakes, made on a 1: 10 and 1:20 scale, were tested in the laboratory under almost the same conditions as in the field.

The short period of operation and certain deviations from optimum conditions of layout and design of the original intake structures do not permit any final conclusions,as yet, on the conditions under which bottom-screen water intakes may be used on the rivers of Kirghizia.

However, the results warrant the following [preliminary] conclusions to be drawn:

1. The insufficient height of the screen sill above the average level of the river bottom reduces the load-transporting capacity of the tailwater section and causes its consequent silting.

2. Insufficient control of streamflow in the area of the water-intake structures, favors the shifting of the river bed and causes uneven distribution of water and silt at the bottom screens.

The stability of flow in the approach reaches of the intake is favored by the curvature of natural or artificial [river] bends, free from large boulder accumulations which divert most of the small-size load from the screens mounted at the river bend, guiding it toward the water outlets.

3. Location of structures on lateral river branches leads to difficulties in the elimination of bed load in both the upper and lower reaches of the intake structures, and requires complicated operations for the transportation of the silted water to the intakes of the bottom screen.

4. Lack of proper correlation between the screen openings, the size grading of the river load, and the load-transporting capacity of the filtering-settling section (gallery, washing channel, settling tank, discharge channel), causes silting and necessitates the manual cleaning of the channels and openings in the load-carrying and discharging sections for the restoration of their discharge capacity.

5. Oblong aprons are liable to break under the impact of large stones. Aprons with deeper blankets and steeper overflow walls below the galleries, covered at their base with small boulders, are the most resistant.

6. The absence of openings in the bottom-screen water intakes for winter supply makes operation of the intake structures less flexible and more

difficult during winter, since, due to freezing, intake of water through the bottom gallery is impossible, even with the screens removed. These difficulties may be alleviated considerably by arranging special and separate water-supply openings adjacent to the screens, provided in front of them with ice-removing flumes.

7. Bottom screens made of round or octagonal bars or of flat iron strips are liable to be clogged by silt; the gaps between the bars widen, and coarse silt penetrates into the galleries. Screens made of coupled angle bars have greater resistance. However, such bars markedly reduce the actual discharge area of the screens and favor the flow of water into the lower reaches (tailwater section) of the intake, along the smooth surface of the angle bars. This is still more enhanced by the excessive slope of the screen (0.14).

8. Screens in which the bars diverge toward the tailwater section are much less likely to clog; the silting is still smaller in screens with bars mounted in the bottom part above the screen frame and not joining it.

A new suggestion of how to solve the problem of clogging is to place round-steel vibrating rods between the screen bars.

9. Screen operation on nonsilting rivers can be improved by providing deep flushing holes (1.5 to 2.0 m wide) between the screen located at the curved bank and the spillway adjoining the opposite convex bank, or by sloping the screen (slope 0.05 to 0.1) from the concave to the convex river bank.

10. Further investigations should be devised for studying the effect of design and location in height of the various structural elements and of channel-shaping processes on the quality of water intakes, in order to determine proper calculation methods, and location and design of such elements.

SCIENTIFIC RESEARCH INSTITUTE OF GIDROPROEKT IMENI S. Ya. ZHUK DEPARTMENT OF WATER-POWER AND ELECTRICAL RESEARCH

Head: U. P. Zavaruev, Engineer

TESTING OF INDUCTION-HEATING OF TRASH RACKS OF HEPS UNDER CONDITIONS OF LARGE FORMATION OF FRAZIL ICE (DURING 1957/1958)

Research by: V. F. Kozlova, Engineer

This report contains the results of investigations carried out during the winter of 1957/1958, of the induction-heated trash racks of the Ak-Kavak-2 HEP of the Ordzhonikidze Cascade Power-plant System of the Tashenergo. It also contains the description of the racks.

The electrical characteristics of the racks were determined, and tests showed that their parameters had been computed with sufficient accuracy.

Trash-rack heating was found to be sufficiently effective to prevent freezing. To ensure normal operation of the trash racks, the exact moment of frazil-ice formation must be correctly determined, and heating started accordingly. Should heating be delayed (during which the reduction in head at the racks will reach 30 cm and more), freezing will become unavoidable.

The paper presents official test reports and blueprints of induction-heated trash racks.

INVESTIGATIONS ON THE USE OF ALTERNATING CURRENT FOR OPERATIONAL PURPOSES

Responsible for Research: V. V. Mikhailov, Candidate of Technical Sciences

Research Team: V. F. Kozlova and E. M. Shtutina, Engineers

This is a report on the results of investigations relating to problems of design, protection, control, and automation of a. c. installations of a large HEP. The feeding circuits of these installations and the central signaling system are described, and the possibility of operating 220 and 500 kv transformer stations on a. c. current is considered. It is shown that automatic control, protection of installations, and the signaling system can operate on alternating current, while the feeding circuits can be reliably supplied from special bus bars.

INVESTIGATIONS ON STATIC OIL-HEATING OF GATE RECESSES IN THE DAM OF THE STALINGRAD HEP

Research by: Beschastnov, Engineer

A static oil-heating system has been designed for heating the gate recesses in the dam of the Stalingrad HEP. Since the calculation of installation of such a heating unit is very difficult, the department for water-power and electrical-engineering research made a special study of the operation of an experimental unit of this sytem.

Results of these investigations are reported, the experimental heating unit and the four heating systems are described, and the volt-ampere characteristics of the heating elements, as well as the typical characteristics of the experimental unit are presented.

The paper also presents conclusions emphasizing the need to vary the oil-heating intensity over the height of the recesses, and to take into account the temperature of heating elements when computing their electric and technical characteristics.

PART VII

INVESTIGATIONS ON HYDRO TURBINE UNITS AND OTHER HYDRAULIC MACHINES

VNIIG IMENI B. E. VEDENEEV LABORATORY OF HYDRO TURBINE UNITS

Head: A. M. Chistyakov, Candidate of Technical Sciences, Senior Research Worker

THEORETICAL AND EXPERIMENTAL SUBSTANTIATION OF THE FORMULA FOR CONVERTING MODEL-TEST DATA TO FULL-SCALE-TEST DATA FOR ALL PARAMETERS (INCLUDING CAVITATION COEFFICIENT) OF HYDRAULIC TURBINES

Responsible for Research: A. M. Chistyakov, Candidate of Technical Sciences, Senior Research Worker

The general research program included the following:

1) development of a method for the more accurate conversion of all the basic parameters of hydraulic turbines from scale-model to full-scale data with allowance for the scale factor;

2) development of methods for deriving the exact theoretical and operational characteristics;

3) development of a method for studying cavitation phenomena on a special test bench with a Venturi measuring chamber;

4) working out a technique for converting the cavitation coefficient from model to full-scale data and determination of operating conditions under which cavitation does not take place in reaction turbines.

To determine the influence of the scale factor on the hydraulic efficiency of the turbine, a general functional relationship for energy losses was derived:

$$\zeta = F\left(\mathrm{Re},\ \mathrm{Fr},\ K_v^*,\ \frac{\Delta}{D_1}\right), \tag{1}$$

where Re = Reynolds number;

Fr = Froude number;

K_v^* = dimensionless velocity coefficient;

$\frac{\Delta}{D_1}$ = relative roughness;

ζ = dimensionless coefficient of the over-all hydraulic resistances which allows for the losses of kinetic energy in the water passages of the turbine and which takes into account both local head losses and head losses due to hydraulic friction.

As a result of model tests on hydraulic turbines (with the Froude number as similarity criterion and with strictest observance of geometrical and kinematic similarity), a curve was obtained giving the dependence of the hydraulic efficiency on the Reynolds number $\varepsilon = f(\mathrm{Re})$, for a turbine operating

under optimum conditions.* It was found experimentally that for a small-scale model of the turbine unit of a HEP (runner diameters $D_1 < 460$ mm) the influence of the scale factor on the hydraulic efficiency, when passing from models to full-scale parameters, is quite pronounced and varies from 3 to 10%.

In large-scale turbine models with runner diameters $D_1 \geqslant 460$ mm and Reynolds numbers of 1.16×10^6 suited to these diameters, the influence of the scale factor on the hydraulic efficiency does not exceed 2 to 3%.

In the year 1957 data were processed of field tests on full-scale hydraulic turbines operating under optimum conditions. The data, obtained from tests on 20 HEPs, both in Russia and abroad, refer to the reaction turbines with speeds ranging from $n_s = 62$ to 710, and in power from 2000 to 83,000 kw. The diameters of the turbine runners ranged from 1.5 m to 9.0 m.

From the test data, a graph was plotted showing a linear relationship $\varepsilon = F(\mathrm{Re})$ with Reynolds numbers ranging from 1.10×10^6 to 70×10^6.

As a result of theoretical research and practical experiments on the influence of the scale factor on the basic parameters of turbines, the following formula for the conversion of the coefficient of hydraulic efficiency from model to full-scale turbines for optimum operating conditions was derived:

$$\varepsilon_f = 1 - [(1 - \varepsilon_m) - 0.0004\,\mathrm{Re} \cdot 10^{-6}], \qquad (2)$$

where ε_f = hydraulic efficiency of a full-scale turbine;

ε_m = hydraulic efficiency of a model turbine with a runner diameter not smaller than 460 mm (the coefficient was obtained in model tests with the Fr number as similarity criterion).

The above graph for the determination of the maximum hydraulic efficiency for reaction turbines as well as formula (2) can be used for turbines provided with standard draft tubes.

From field tests on hydraulic turbine units it was found that, by plotting the turbine characteristic from the coefficient of hydraulic efficiency $\varepsilon_m = f(n_1'^*, Q_1'^*)$ instead, as is usually done, from the full efficiency $\eta_m = f(n_1', Q_1')$, it is possible to obtain, without any conversion and with sufficient accuracy, the series characteristics $n_1'^* = \frac{n_1' D_1}{\sqrt{\varepsilon H}}$ and $Q_1'^* = \frac{Q}{D_1^2 \sqrt{\varepsilon H}}$ for all the operating conditions of full-scale turbines.

Thus, the method for plotting the hydraulic-efficiency and full-efficiency characteristics of a turbine, described in more details in our technical-information bulletin (Gosenergoizdat 1958), is of great practical importance and permits the designer the selection of the most effective sets of basic parameters for reaction-turbine units.

During the years of 1957 and 1958 a study concerning the conversion of the cavitation coefficient from the model to the full-scale units was carried out. This was done by determining the condition for cavitationless operation of the reaction turbine. We devoted considerable time to the study of the physical process of cavitation, its determining factors, and the mechanism of cavitational erosion of materials involved.

* Chistyakov, A. M. Novaya metodika model'nykh issledovanii turbin reaktivnogo tipa i gidroturbinnykh blokov GES (A New Method of Scale-model Research on Reaction Turbines and on Hydro-Turbine Units in HEPs). -Gosenergoizdat. 1958; see p. 27, Fig. 6; p. 30, Fig. 8.

On the basis of these studies, the following general functional expression connecting cavitation with its determining factors was derived. The cavitational coefficient is thus written:

$$\sigma_{cr} = \Phi\left[\frac{VL}{\nu}, \frac{\Delta P}{\rho V^2}, \frac{V^2}{gL}, \frac{V\sqrt{D}}{\sqrt{\frac{S}{\rho}}}, \delta_0, \frac{fl}{V}, \frac{Vl}{a_t}\, \frac{r}{C\Delta t}, \frac{V}{\sqrt{2g\varepsilon H}}, \frac{\Delta}{D}\right] = \tag{3}$$

$$= \Phi\left[\mathrm{Re}, \mathrm{Eu}, \mathrm{Fr}, \mathrm{We}, \delta_0\, C, \mathrm{Pe}, \mathrm{Ku}, K_v^*, \frac{\Delta}{D}\right],$$

where Re = Reynolds number;
Eu = Euler number;
Fr = Froude number;
We = Weber number;
δ_0 = coefficient allowing for air entrainment into the working part of both model and full-scale water passages;
$C = \frac{fl}{V}$ = coefficient, allowing for the conditions of the compression shock propagating with supersonic speed on the border of the cavitation area;
Pe and Ku = Peclet and Kutateladze numbers which are similarity criteria of heat exchange during the collapse of the cavitation bubble or of the cavitation cavity;
K_v^* = criterion of kinematic similarity (i. e. similarity of velocity triangles);
$\frac{\Delta}{D}$ = criterion of geometrical similarity (relative roughness).

The general functional dependence of the cavitation coefficient (3) on dimensionless parameters shows that the role of scale effect in the phenomena of cavitation must be investigated by physical means with the simultaneous widening of the scope of investigations into the physical nature of cavitation.

At the present state of our knowledge of cavitation the conversion of the cavitation coefficient (σ) from model to the full-scale data can be done using the formula suggested by Professor V. S. Kvyatkovskii:

$$(\sigma_{\mathrm{turb}})_f = \frac{\eta_{h.f}}{\eta_{h.m}}(\sigma_{\mathrm{turb}})_m + \frac{\eta_{h.f}}{\eta_{h.m}}\left(\frac{\eta_{h.f}}{\eta_{h.m}} - 1\right) \times$$

$$\times\, \eta_{d.m}\eta_{h.m}K^2_{v,\mathrm{av}} + a_m\left(\frac{\eta_{h.f}}{\eta_{h.m}}\frac{D_m}{H_m} - \frac{D_f}{H_m}\right), \tag{4}$$

where $\eta_{h.f}$ = hydraulic efficiency of a full-scale turbine;
$\eta_{h.m}$ = hydraulic efficiency of a model turbine;
$\eta_{d.m}$ = efficiency of the draft tube;
$K_{v,\mathrm{av}}$ = velocity coefficient;
a_m = coefficient determining the point of minimum pressure in the stream.

The conditions for the cavitationless operation of a turbine can be determined by the following inequalities:

$$\left.\begin{aligned}\sigma_u &> \sigma_{\mathrm{turb}}\\ P_{\min} &> P_{i.p}\end{aligned}\right\}, \tag{5}$$

where $\sigma_u = \frac{H_a - H_s - H_{i.p}}{H}$ = cavitation coefficient of the turbine unit (determined according to the Thoma formula).

For investigation on cavitation and cavitational erosion, ultrasonic tests on a cavitational test rig of the VNIIG with Venturi measuring chamber were used.

In the year 1958 cavitation was investigated by means of ultrasonic tests on the turbines of the Narva and the Pal'eozero HEP.

As a result of these investigations and of tests on the VNIIG-KA-1 cavitation analyzer, the following method for investigating cavitation in the turbines of the Kakhovka HEP was suggested.

1. Field tests of cavitation in operating turbines is carried out by means of the ultrasonic method in which the VNIIG-KA-1 cavitation analyzer records the beginning and the development of unsteady local cavitation in turbines.

2. A barium-titanate piezoelectric transducer located in the turbine cover serves as a pick-up for the ultrasonic vibrations arising in the stream during the occurrence of cavitation in the hydraulic turbine.

3. The intensity of cavitation, taking place under various operating conditions of the turbine, is determined from the following parameters:

a) the high-frequency spectrum of cavitation plotted as typical selective curves preliminarily rated under laboratory conditions. The frequency range of these spectra lies between f_{min} = 27.4 kc and f_{max} = 1570.0 kc;

b) the amplitude of ultrasonic vibrations at the resonant frequencies, as measured by a microammeter connected across the cavitation analyzer bridge.

c) visual observation on a cathode-ray oscilloscope of cavitational phenomena occurring in the water passages of the turbine;

d) study of the oscillograms obtained from the oscilloscope connected to the cavitation analyzer.

4. For each experiment made under given operating conditions of the hydro unit, a curve is plotted expressing the relationship between the amplitude of ultrasonic vibration and the spectrum of cavitational frequencies $[A = F(f)]$. From these curves the amplitude of the main resonant frequencies for various working conditions of the hydro aggregate is then found and expressed as a function of the following parameters: power measured at the generator terminals $A = F_1(N_a)$, vacuum in the draft tube $A = F_2(V_{d.t}$, and in the case of an adjustable blade turbine, blade-opening angle $A = F_3(\varphi°)$

The intensity of cavitation under various operating conditions of full-scale turbines is found from the analysis of all the above-mentioned relations.

In the second stage of the experimental program, high-speed motion-picture photographs (4000 frames/second) were taken of the process of formation and collapse of the cavitational bubbles.

The above experiments, and the analysis of the motion picture confirmed the correctness of Einstein's assumption that at the border of the cavitation area develops the so-called compression shock which is related to the process of formation and closure of the cavitation hollows. According to Einstein, the cavitation area in the fluid corresponds to the area of supersonic velocities.

The cavitation experiments, performed both on model and full-scale turbines, showed that the ultrasonic method is best suited to the study of local unsteady cavitation in the test flow chamber, or in the water passages of a turbine. Local unsteady cavitation is characterized by the development of supersonic processes and may be described by the dimensionless criterion

$C = \frac{fl}{V}$, where f is the sound-wave frequency.

OPTIMUM DESIGN AND EXPERIMENTAL INVESTIGATION OF THE WATER PASSAGES OF STANDARD HYDRO TURBINE UNITS FOR RUN-OF-RIVER HYDRO POWER PLANTS

Responsible for Research: A. M. Chistyakov, Candidate of Technical Sciences, Senior Research Worker

Research by: I. V. Plokhotnikov, Engineer

In order to ensure optimum water-flow conditions in both the scroll case and distributor of hydraulic turbines, research was undertaken to design scroll-case flow-dividing piers of proper shape suited to such optimum conditions. The research schedule also provided for the design, construction, and installation of the runner $D_1 = 250$ mm of the model PL-661-25 Kaplan turbine, together with a special braking device intended for the aerodynamical testing of all the water-passage elements of turbines for run-of-river hydro plants.

In conformity with the research schedule, in 1958, the following stages were carried out:

1) design, construction, and installation on the aerodynamical test stand, of the model turbine type PL-661-25 (runner diameter $D_1 = 250$ mm) provided with a braking device;

2) aerodynamic investigation of scroll cases having an enveloping (nose) angle $\beta = 180°$, and various shapes and over-all sizes;

3) investigation of the water-flow pattern at the entrance into the speed ring and behind the distributor, for a scroll case provided with one or two dividing piers, having an enveloping angle $\beta = 180°$ and an internal width $W_{in} = 1.2D_1$;

4) processing and interpretation of the experimental results and their presentation as curves of distribution of peripheral and radial flow-velocities of the stream flowing around the turbine speed ring (Figures 226 and 227).

Analysis and comparison of the design variants investigated in terms of distribution of peripheral and radial flow velocities at different distances of the dividing pier heads in the scroll case from the runner axis, permit the following conclusions.

1. The design shown in Figure 228 ensures more uniform distribution of radial and particularly peripheral components of velocity along the speed-ring circumference than the other variants presented.

According to this design, the scroll case with an enveloping angle of 180° and an internal width of $W_{in} = 2.5\ D_1$ (of the type used at the Votkinsk HEP) has a single dividing pier located at a distance of 10 m from the turbine-unit center line (at the model we selected a distance of 264 mm at a runner diameter $D = 250$ mm and a scale ratio $\lambda = 37.2$)

2. Optimum shape of the dividing-pier head is ensured by the following dimensions (see Figure 228):

R_1 = 435 mm (1.74 D)
R_2 = 177 mm (0.71 D)
R_3 = 10 mm (0.04 D)
a_1 = 478 mm (1.912 D)
b_1 = 147 mm (0.59 D)
a_2 = 136 mm (0.545 D)
b_2 = 154 mm (0.616 D)

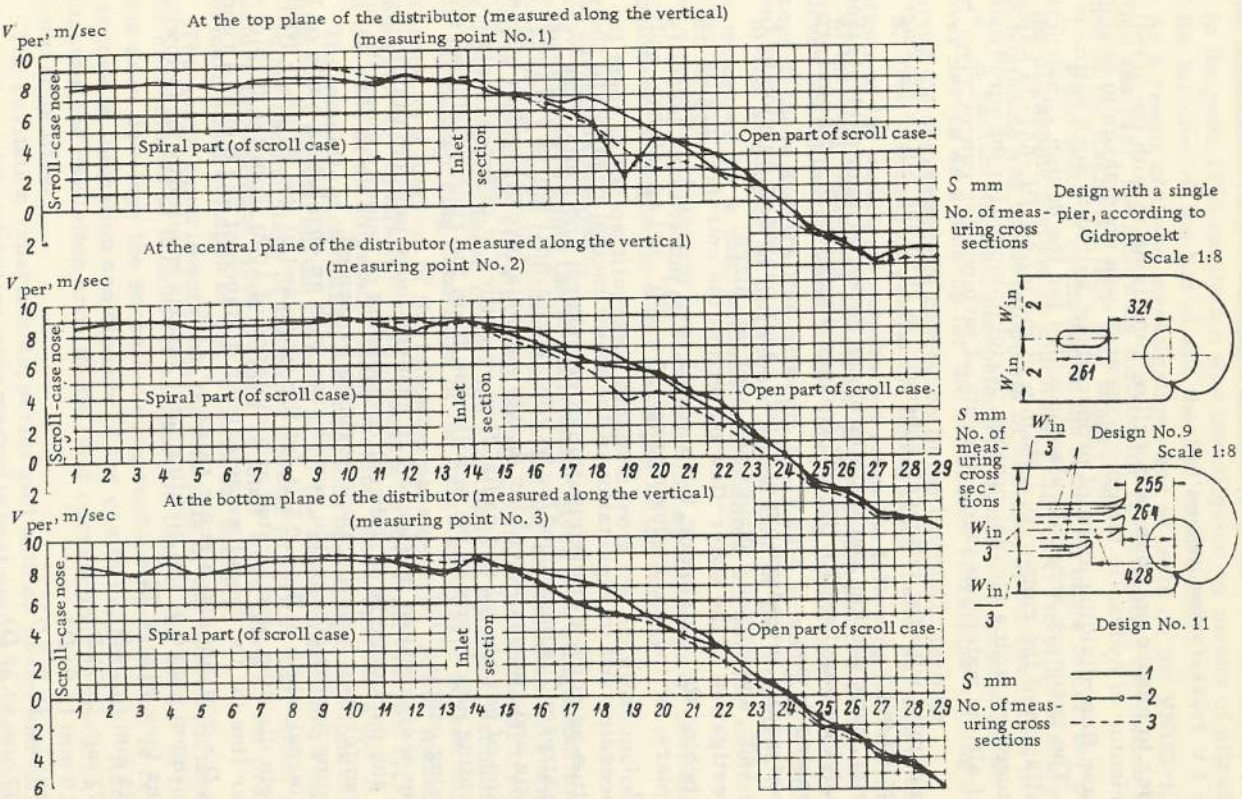


FIGURE 226. Distribution of peripheral speed-ring velocities $v_{per} = v \cos \delta \sin \varphi$ along the circumference

1–scroll case (SC) with two piers, design No. 9; 2–scroll case (SC) with one pier, design No. 11; 3–scroll case (SC) according to the Gidroproekt design.

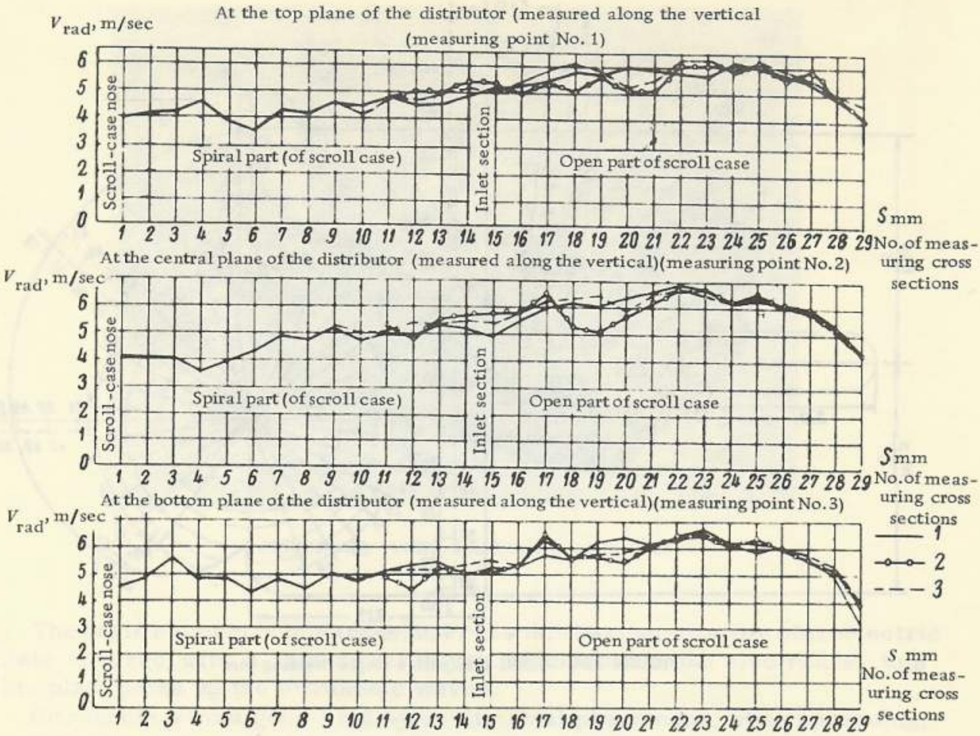


FIGURE 227. Graphs showing distribution of radial velocities $v_{rad} = v \cos\delta \cos\varphi$ along the circumference of the speed ring

The results of these investigations, and a comparative analysis of the flow-velocity distribution in scroll cases with one dividing pier (Figure 228), in scroll cases with two dividing piers (Figure 226, design No. 9), as well as in scroll cases with a single pier designed by Gidroproekt for the Votkinsk HEP, show that for a standard turbine bay (block) as used at this power plant, the pier design shown in Figure 228 is to be recommended.

Design No. 9 in Figure 226 having two dividing piers may be recommended for the layout of the turbine bay. This design ensures the following values for the distance of the piers from the center line of the turbine units: l_1 = 428 mm (1.71D) and l_2 = 255 (1.02D). The recommendations given in this study are only tentative. For final conclusions, a series of new, more detailed investigations scheduled for the year 1959 will involve aerodynamical model tests of the scroll cases with dividing piers together with the type PL-661-25 model turbine installed in the turbine bay of the Votkinsk HEP.

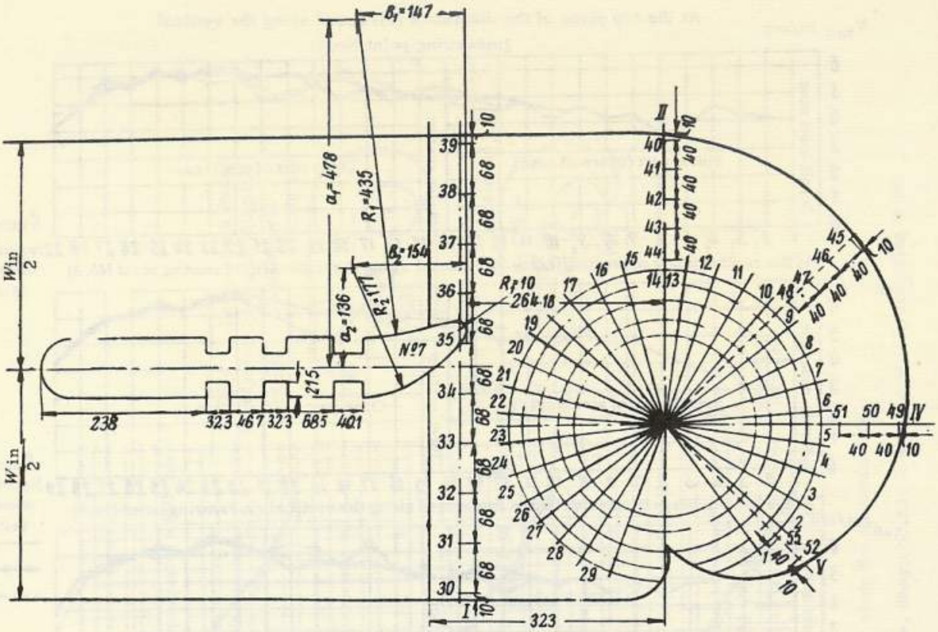


FIGURE 228. Scroll case with a single dividing pier

FIELD INVESTIGATIONS ON CAVITATION IN THE TURBINE UNITS OF THE KAKHOVKA HEP

Responsible for Research: A. M. Chistyakov, Candidate of Technical Sciences, Senior Research Worker

Research by: I. I. Ivanov, Senior Engineer

The aim of this study was to investigate the intensity of cavitation occurring in hydraulic turbines under field conditions.

These investigations were carried out under load conditions specially selected when checking the regulating connections between the guide vanes, the blade-control valve, and the blade-rotating mechanism, by the relative-efficiency method.

The investigations were conducted on three turbines which were tested under 148 different operating conditions.

The ultrasonic method, developed by the VNIIG, was used as the basic method for cavitation investigations of hydraulic turbines under field conditions.

According to this method, applied for the first time at the Kakhovka HEP, a special cavitation analyzer of the VNIIG-KA-1 type (Figure 229) records the intensity of ultrasonic vibrations generated by the continuous high-frequency pressure fluctuations in the water passages of the turbine during cavitation.

The intensity of these ultrasonic vibrations was measured by a piezoelectric transducer installed on the cover plate of the turbine.

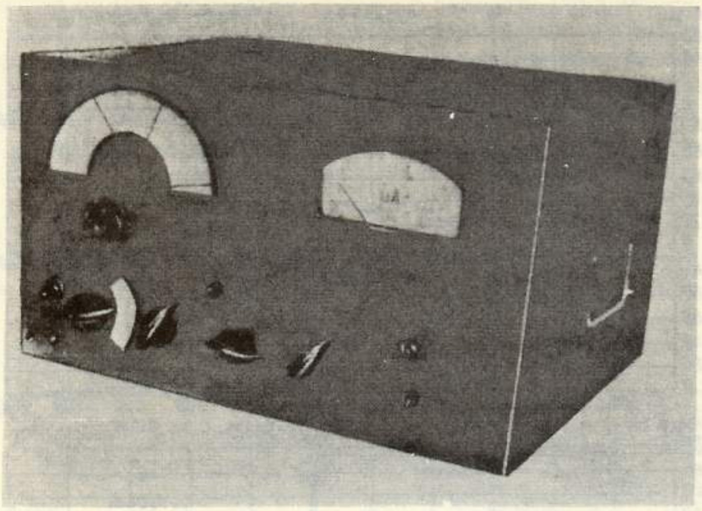


FIGURE 229. VNIIG-KA-1 cavitation analyzer

The basic element of this transducer is a thin barium-titanate piezoelectric plate covered with a thin silver layer which forms the electrodes; this thin plate picks up the ultrasonic waves.

Ultrasonic waves generated by cavitational phenomena, when spreading, encounter the piezoelectric transducer where they are transformed into electric pulses [proportional to the intensity of the ultrasonic waves].

The variable electric pulses are fed to the cavitation analyzer. The latter is of the wide-band selective-circuit type, responding to the whole frequency spectrum of the ultrasonic waves generated by cavitation.

Intensity (and hence the amplitude of ultrasonic waves) of cavitation is measured by a microammeter connected across the measuring bridge. A special oscilloscope of the EO-7 type connected in the circuit reproduces visually the pattern of cavitational vibrations.

As a result of cavitational investigations, the function $A = F(f)$ for the whole frequency spectrum of the ultrasonic waves is plotted (see Figure 230).

A number of 148 oscillograms were obtained for all the tests. These diagrams permitted the plotting of the following functions:

$$A = F_1(N_a);\ A = F_2(P_{\text{vac. dr. t}}) \text{ and } A = F_3(\varphi^\circ).$$

The most typical of all these functions is $A = F_3(\varphi^\circ)$ (see Figure 231) as it shows the intensity of cavitation in Kaplan turbines.

In this diagram the amplitude of the cavitational ultrasonic vibrations is plotted on the ordinate, and the blade-opening angle, on the abscissa.

A close scrutiny of this diagram reveals the following:

1. Cavitational conditions differ with the turbine type.
2. Maximum amplitude of cavitational vibrations is noticed at blade-opening angles $\varphi = 15$ to $17°$.

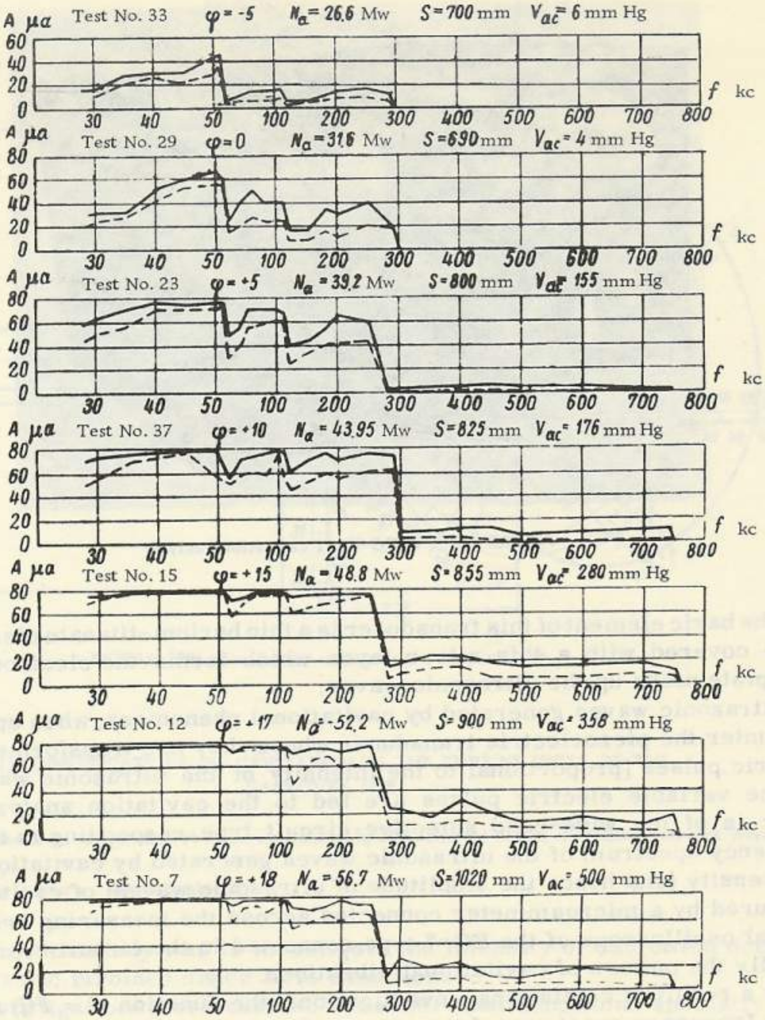


FIGURE 230. Oscillogram of cavitational vibrations recorded at the turbine unit No. 6 of the Kakhovka HEP

As a result of cavitational investigations at the Kakhovka HEP, the following conclusions may be drawn:

a) The method for cavitational investigations of hydraulic turbines as developed by the VNIIG proved its suitability and may be recommended for field investigations.

b) The VVNIIG-KA-1 cavitation analyzer permits the recording of cavitation intensity of varying amplitude.

c) Processing of cavitational oscillograms according to these methods gives both qualitative and also certain quantitative characteristics of the cavitation phenomena recorded. Thus, for instance, the increase in intensity of draft-tube and runner-blade cavitation (for the Kakhovka turbines) is

characterized by the appearance of higher resonant frequencies; whereas with [normal] load, blade-opening angles varying from $\varphi = -5$ to $\varphi = +10°$, and power output at the generator terminals varying from 20 to 45,000 kw, the frequency spectrum is limited to a range of 300 to 500 kc. With large blade angles (φ = 15, 17, 18, and 19°) and a power output higher than 45,000 kw the oscillation frequency markedly increases (up to 750 kc) thus increasing the erosive wear of the water-passage elements (runner blades and draft tube).

d) The use of the VNIIG-KA-1 cavitation analyzer permits the investigation and proper adjustment of the blade-control valve mechanism which regulates the runner-blade opening (runner-blade angle).

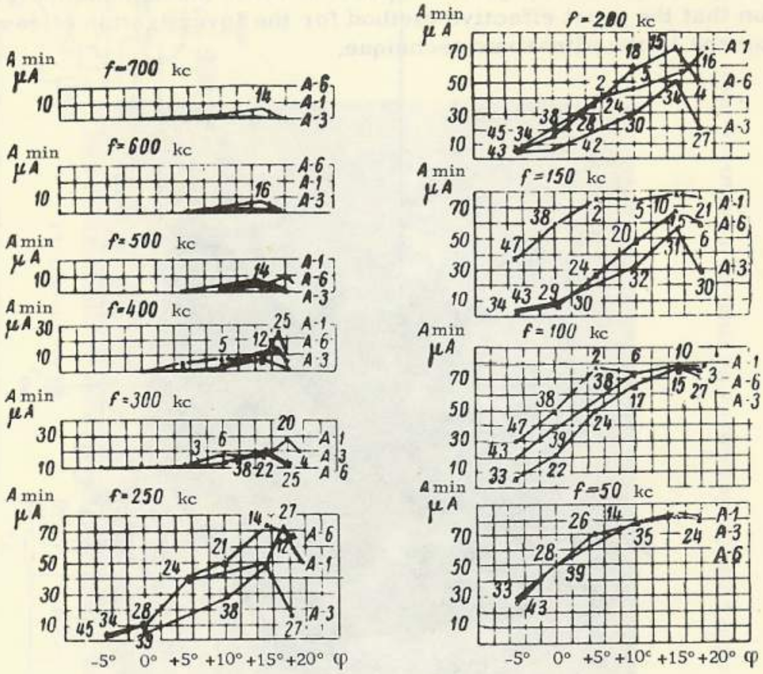


FIGURE 231. Cavitation intensity vs. blade-opening angle $A = F_3(\varphi°)$

DESIGN AND CONSTRUCTION OF DEVICES FOR CAVITATION RESEARCH AND UNDERWATER INVESTIGATION OF THE STATE OF HYDRO STRUCTURES

Responsible for Research: I. I. Ivanov, Senior Engineer

Research by: A. S. Lashkov, Engineer

The study was carried out in two stages.

The program of stage A provided for the running-in of the cavitational analyzer designed by the laboratory, the design of an ultrasonic-wave

receiver, construction of these devices, and their use both in model and field tests.

Stage B envisaged the design and construction of special devices for underwater investigations of hydro structures.

Within the framework of stage B the researchers studied in 1958 the existing designs of remote-indicating devices used both in industry and oceanography.

Most attention was focused on problems studied in stage A. The theoretical concepts and propositions underlying the design of the cavitation analyzer for hydro turbines were based on the fact that the development of cavitation causes in the stream a local raise in pressure similar in its nature to the water hammer.

The analysis of the physical aspects of these phenomena led to the conclusion that the most effective method for the investigation of cavitational phenomena is the ultrasonic technique.

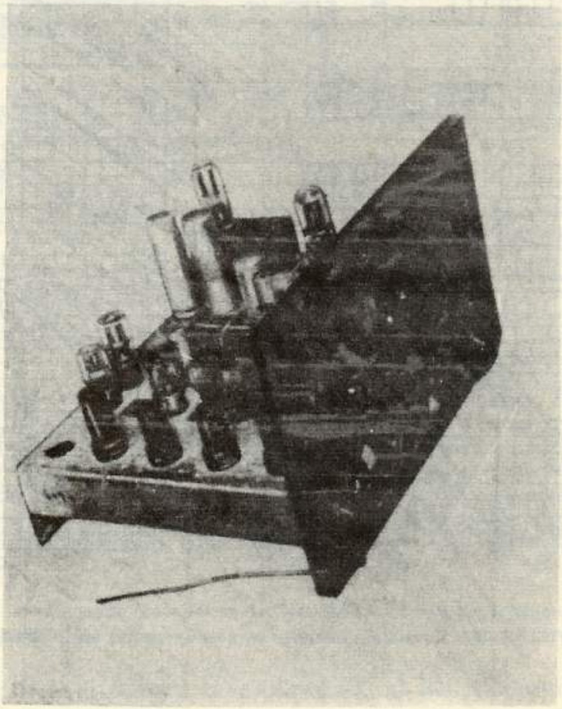


FIGURE 232. VNIIG-KA-7 cavitation analyzer

The basic principle of the cavitation analyzer investigated is the frequency principle which makes it possible to investigate as large a spectrum of cavitational frequencies as possible.

Any type of piezocrystalline plates (quartz, turmaline, Seignette-Rochelle salt, barium titanate, etc.) may be used as ultrasonic transducer for the pressure fluctuations appearing in the stream subject to cavitation. The

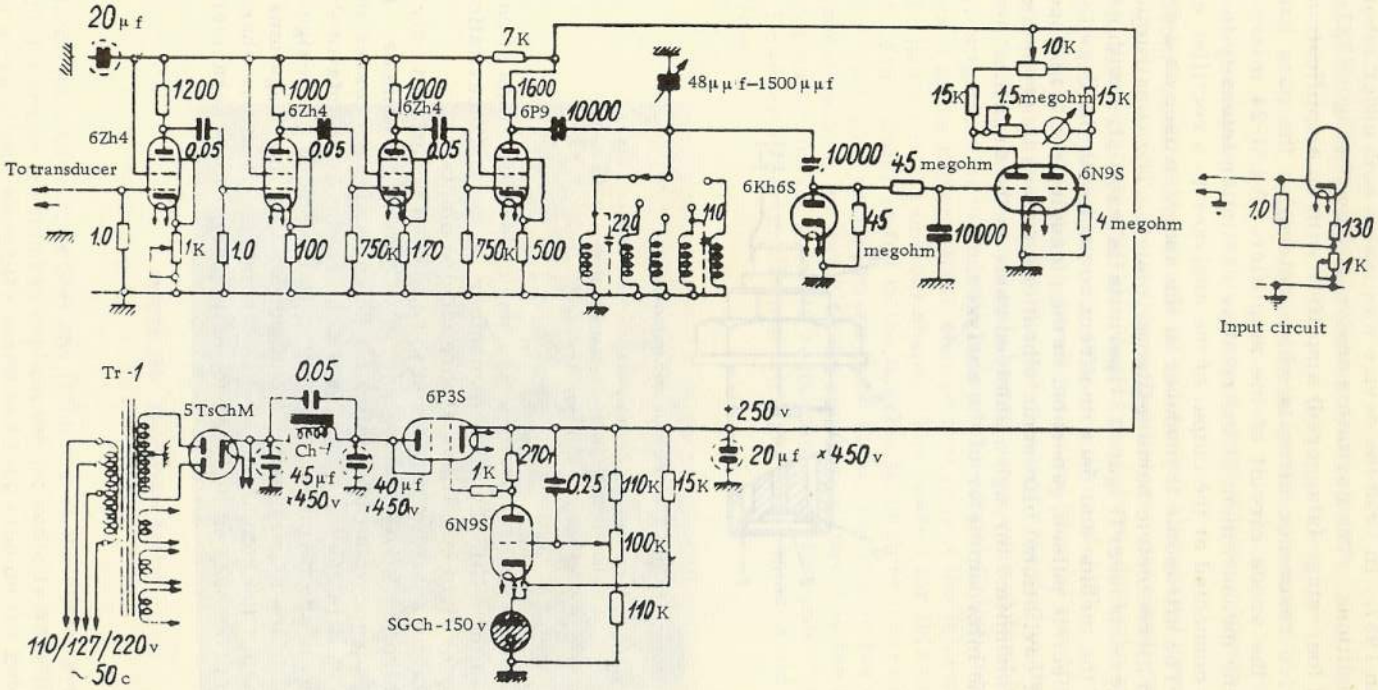


FIGURE 233. Circuit diagram of the VNIIG-KA-1 cavitation analyzer

VNIIG-KA-1 cavitation analyzer (Figure 232) was designed and built by the laboratory in 1957. In 1958 the device was tested both under laboratory and field conditions. The cavitation analyzer shown in Figure 233 is actually a wide-band four-stage (staggered) amplifier with an amplification factor of 1000. Five resonance circuits which determine the pass band are included in the anode circuit of the amplifier. An M-24 microammeter, permitting the measurement of the relative cavitation intensity in milliamperes, is connected at the output of the analyzer to a rectifier and a d. c. amplifier. The ultrasonic transducer of the analyzer uses as a sensing element thin piezoelectric barium-titanate plates. The construction of the piezoelectric transducer (Figure 234) permits its installation within the water passages of the turbine and thus its direct contact with the "cavitating" medium. Electric pulses, generated in the piezoelectric transducer under the action of cavitational ultrasonic vibrations, caused by pressure fluctuations, are transmitted through a shielded cable to the amplifier and recorded by the microammeter of the analyzer.

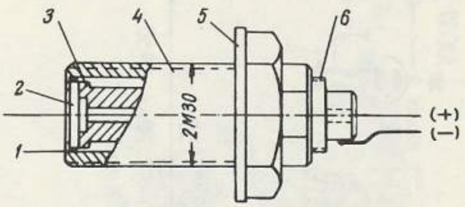


FIGURE 234. Piezoelectric transducer:

1–rubber ring; 2–piezoelectric sensing element (thin barium-titanate plate); 3–textolite sleeve; 4–transducer body; 5–lock nut; 6–plug.

The new device made it possible to conduct investigations of cavitation phenomena, in the VNIIG cavitation chamber, on the LMZ cavitation test stand, and under field conditions at three hydro plants.

The cavitation analyzer was found to be able to record not only the beginning but also the whole development of cavitation phenomena. From the amplitude-frequency characteristics of the cavitation processes in full-scale turbines, it is easy to conclude which are the most unfavorable operating conditions of the turbines. Further refinements in the design of the analyzer and in the experimental technique for field investigations will undoubtedly permit the researcher to pass from the qualitative characteristics of cavitation processes in turbines to their quantitative characteristics.

TNISGEI IMENI A. V. VINTER HYDRAULIC-ENGINEERING LABORATORY

Head: L. G. Gvelesiani, Candidate of Technical Sciences, Lecturer

MODEL INVESTIGATIONS OF PELTON TURBINES

Responsible for Research: G. Kh. Tkhinvaleli, Candidate of Technical Sciences, Senior Research Worker

In connection with operational deficiencies revealed upon inspection of the Pelton turbines delivered by a Swedish company and installed at the Khrami HEP, the TNISGEI decided to conduct model tests, and designed for this purpose a special test rig (Figures 235 and 236).

The analysis of turbine operations at the Khrami power plant showed that the probable reason for the deficiencies revealed is the spontaneous deflection of the stream jet as a result of its conical terminal shape.

Investigations of the needle nozzle of a model Pelton turbine similar to that used at the Khrami HEP, showed that in fact the jet, emerging from the nozzle, has a certain conical shape. The reason for this conicity had to be found. Since water is fed to the nozzles through several closely located elbows — a layout which proved to be highly unsuitable (as shown by numerous investigators), we came to the conclusion that it is the system of incorrect water intake which is responsible for the conical deflection of the jet. However, experiments with straight water intakes did not lead to positive results.

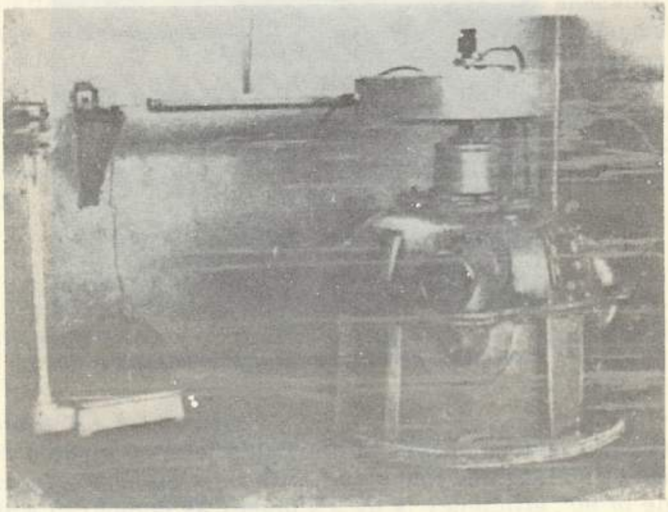


FIGURE 235. Model Pelton turbine

Since we could not reveal any influence of the elbow-system water intake on the jet shape, we tried to explain the jet conicity as being caused by the eight streamlined, carefully machined long ribs on the guide bushing in the immediate vicinity of the nozzle.

Our investigations actually confirmed the existence of a certain conicity of the water jet emerging from the nozzle as a result of which, at large openings of the nozzle, the water jet misses the buckets, producing no useful work. We therefore decided to straighten the jet by redesigning the nozzle and its needle. Three sets of nozzles and needles of different profile were designed and constructed. For greater accuracy the tests were conducted with a single nozzle (with straight-line water intake) and at a head of 50 m.

Investigations showed that, with a nozzle-tip angle of 90°, the jet emerging from the nozzle tip is more concentrated compared with the jet emerging from nozzles and needles of other shape.

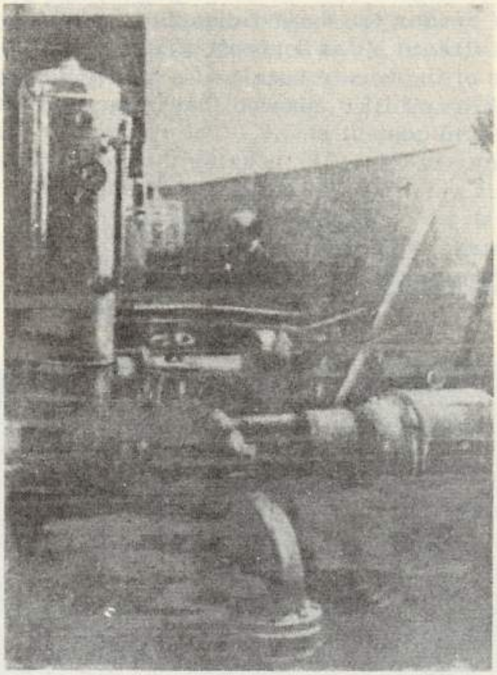


FIGURE 236. Intake structures of the model-turbine unit

To compare various shapes of emerging jets, we studied their effect on the turbine efficiency and found that, with a jet emerging from a nozzle with a tip angle of 90°, efficiency is by 3% higher as compared with jets emerging from conventional nozzles. Moreover, the jet emerging from a nozzle with a tip angle of 90°, is more concentrated (i.e., less conical).

Investigations on a model-turbine unit showed that the deficiencies noticed at the Pelton turbines of the Khrami HEP may be eliminated by changing the design of the nozzle and needle.

GIDROPROEKT IMENI S. Ya. ZHUK DIVISION FOR HYDRO POWER AND ELECTRIC RESEARCH

Head: V. P. Zavaruev, Engineer

THE EFFECT OF CAVITATION ON TURBINE BLADES OF HYDRO UNITS OPERATING UNDER CONDITIONS OF INCREASED POWER GENERATION

Responsible for Research: B. A. Bal', Engineer

Research by: Z. N. Gur'eva, Engineer

The results of field tests conducted on the turbine units of the Volga HEP imeni V. I. Lenin showed that the turbines permit operation under load conditions higher than those indicated in the engineering specification of the LMZ — the manufacturer of these turbines. To check whether these conclusions are achievable in practice, it was necessary to test the cavitational behavior of turbines under increased load. So far there are no established methods for testing the cavitational properties of turbines under field conditions, and the existing laboratory technique is not applicable under the given conditions. To solve these problems, a special method was developed, the so-called rapid-erosion method, which permits the establishment of the possible limits of load increase by the extent and intensity of erosive failure of the turbine blades under the action of cavitation. In this method metallic plates with low resistance to cavitational erosion are subjected to the action of cavitation. The test plates were made of pure, annealed aluminum sheets of grade ADIM and tested on a large PL-587-100 model turbine at the Skhodnya hydraulic-test station under conditions simulating, as far as possible, field conditions. The test results made it possible to establish the possible limits within which the power output of the turbine may be safely increased and the results were used by the Ministry of Electric Power Plants to substantiate its decision on the increase of the power output of the Volga HEP.

EXPERIMENTAL INVESTIGATION OF THE E-10 ELECTROHYDRAULIC TURBINE GOVERNOR SUPPLIED BY THE KMV-ASEA COMPANY

Responsible for Research: B. A. Bal', Engineer
L. A. Pavlov, Engineer

Research Team: B. A. Kovrigin, Engineer
I. V. Mezhuev, Engineer
B. E. Mitrofanov, Engineer
Ya. A. Gradil, Engineer

Experimental investigations of the E-10 turbine governor were carried out on the large-model turbines of the Skhodnya hydraulic-test station.

The results of these investigations showed that:

1) the use of electric [and electronic] equipment facilitates a relatively easy widening of the range over which separate parameters of the governor

(e. g., its damping) may be adapted to variation in operating conditions, a fact of particular importance for governor tuning-in during operation;

2) the use of an electronic amplifier permits the reduction of the power of control-parameter transducers and a simple solution to the problem of summing up these parameters;

3) the reliability of governor operation is ensured by the high constructional quality of component elements and automatic protection devices of the governor tested;

4) the design of the governor housing, console and switchboard ensures easy access to all the governor parts.

POWER AND VIBRATION FIELD TESTS OF THE TURBINE UNITS OF THE VOLGA HEP IMENI V. I. LENIN

Responsible for Research: M. F. Sarkisova, Engineer
L. N. Goncharov, Engineer

Research Team: S. I. Gorbachev, Engineer
E. I. Pertsova, Engineer
D. B. Radkevich, Engineer
G. M. Turnyanskii, Engineer

During operation of the turbine units of the Volga HEP imeni V. I. Lenin, it was found that they permit an increase in power output [over the limits established in the engineering specifications].

A series of field tests were carried out to study the practical possibilities for realization of these conclusions without exceeding the vibration resistance of the turbines, and to evaluate the operation of the guide-vane – runner-blade control connection from the relative-efficiency values. The tests were conducted over a wide range of head fluctuations at the turbine units Nos 3, 5, and 7.

The following conclusions were drawn from the tests.

1. The possibility was confirmed of increasing the power-output limits of the turbine units. Thus, e. g., at a head range of 17 to 22 m and runner-blade angles smaller than 17 – 19°, which were established by the manufacturer in the limits for the guide-vane – blade-control connection, the power output of each turbine unit may be safely increased by 2 – 6,000 kw and the vibrations of the turbine cover plates for various operating conditions can be reduced.

2. To obtain maximum efficiency in operation of the turbine units installed at the plant, it is necessary to improve the guide-vane – blade-control connection and the operational characteristics, by carrying out tests on a turbine model with a runner diameter of 1 m both at the Skhodnya test station and in situ.

OPERATIONAL BEHAVIOR OF A KAPLAN TURBINE UNDER CONDITIONS OF OPENING OF THE GUIDE VANES TO NEGATIVE-ANGLE VALUES

Responsible for Research: S. I. Gorbachev, Engineer

Research by: Z. N. Gur'eva, Engineer

The paper deals with the problem of reducing the runaway speed of turbines by opening the guide vanes to negative angles.

The tests were carried out at a large test stand on a PL-589-100 Kaplan-turbine model, whose distributor has 32 guide vanes of symmetrical profile and a height (length) of $0.41D$. The circle of the guide-vane rotation axes has a diameter of $1.2D_1$.

The tests were conducted at a head range of 10 to 28 m.

The test results showed the possibility of reducing the runaway speed of turbines by opening the guide vanes to negative angles.

STUDIES ON THE POSSIBILITY OF DESIGNING HYDRAULIC TURBINES WITHOUT ALLOWANCE FOR RUNAWAY SPEED

Research by: V. V. Mikhailov, Candidate of Technical Sciences

One of the ways to reduce construction cost and to increase the efficiency of hydro power plants is the increase of the power output of the turbine units.

But the increase in power output of the turbines is limited by mechanical-strength requirements of the generator rotor to allow for runaway speeds.

Until recently, generator rotors coupled with Francis turbines were designed for a runaway speed $(1.7 - 1.8)n_{\text{nom. turb.}}$ and generators coupled into Kaplan turbines, for a runaway speed of $(2.1 - 2.2)n_{\text{nom. turb.}}$

The problem of such high allowances for rotor speeds and the methods for their reduction are still waiting for a suitable solution.

The study deals with causes of failures, and experience in operation of hydro power plants of the U. S. S. R.

Analysis of operational data showed only few cases of rotor runaway conditions; runaway occurred in small-size units not equipped with automatic speed control. The requirements as put forth by the specifications of rotor design for runaway speed have no engineering or economic substantiation. Operation of hydraulic turbines was studied without any consideration of actual operating conditions of the whole unit, without any interrelation with; the electric equipment. Also no consideration was given to the fact that the unit may fall into runaway speed only upon coincidence of several abnormal, operational factors (sudden disconnection of the unit from the high voltage transmission lines, failure of the speed-control system of the runaway protection, etc.). An estimate of the frequency for such a coincidence to occur, according to statistical data on failures and shutdowns in the hydro plants belonging to the Ministry of Electric Power Plants, shows that the probability for runaway to occur is one case in 20 to 25 years for 500 hydro units.

Technical-economical analysis shows that the savings in yearly expenditure on maintenance of turbine units and power plants that might be obtained by reducing the design speed [of runner and rotor] markedly exceeds the losses incurred as a result of rare cases of runaway.

On the basis of this analysis, the suggestion was made to reduce the rated runaway speed to $(1.6 - 1.65)n_{\text{nom. turb.}}$ for Francis turbines and to $(1.6 - 1.75)n_{\text{nom. turb.}}$ for Kaplan turbines.

PART VIII

HYDROTECHNICAL STRUCTURES OF THERMAL POWER PLANTS

LABORATORY OF INDUSTRIAL HYDRAULICS VNIIG IMENI B. E. VEDENEEV

Head: A. G. Averkiev, Candidate of Technical Sciences, Senior Research Worker

ICE-THERMAL LABORATORY

Head: Professor A. M. Estifeev

STUDY OF THE HYDROTECHNICAL STRUCTURES OF COOLING UNITS FOR WATER-SUPPLY SYSTEMS OF THERMAL POWER PLANTS

Responsible for Research: For Part I: G. V. Vostrzhel, Candidate of Technical Sciences, Senior Research Worker
A. I. Pekhovich, Candidate of Technical Sciences, Senior Research Worker
For Part II: E. K. Trubina, Postgraduate

The study, carried out jointly by the laboratories for industrial hydraulics and for ice-thermal conditions, deals with problems in the design of cooling structures for water-supply systems of thermal power plants.

The investigation was carried out in two stages, involving:

1) the improvement of hydrothermal calculation methods for cooling ponds;

2) the study of water exchange between the inflowing stream and the vortex zones in the pool.

The first part includes theoretical and experimental studies.

Theoretical study

1. An extensive review of the methods used for the hydraulic calculations of cooling ponds was effected. According to the new calculation methods, the streamflow widening can be attributed to the joint action of three resistance forces: bottom-friction forces, tangential forces, and hydrostatical-pressure forces.

The analysis of the results of the calculations show that the influence of the bottom-friction forces and of the forces of tangential stresses varies with the flow conditions. When the numerical value of the ratio B_0/h is small (the ratio of the width of the inflow to its average depth), overlooking the influence of tangential stresses can result in a considerable decrease in the calculated values of the transversal dimensions of the inflowing

stream, and vice versa, when the ratio $B_0/h > 25$, the determining factor is the bottom-friction force (Figure 237). Therefore, the limit for using either of these methods will be $B_0/h = 25$. In this case, the error in the calculated width of the outflowing jet made by considering only the bottom-friction forces will be below 15%.

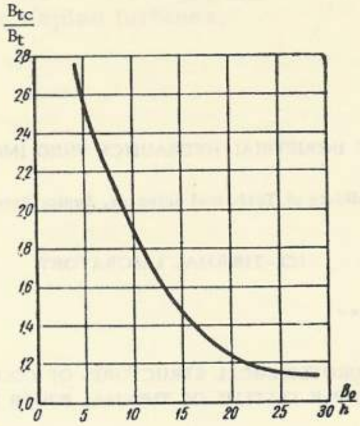


FIGURE 237

2. The problem was studied of the influence of various kinds of heat transfer (molecular, free convection, and turbulent convection on the thermal state of the inflowing stream.

It has been found that the heat transfer in the direction of the flow decreases the efficiency of the cooling basin. Practically, this factor has such a small influence that it may be ignored. On the other hand, heat transfer normal to the direction of flow improves the efficiency of the cooling basin. The influence of this factor can be very considerable.

3. The relationship between surface, bottom temperatures, and the Biot number (Bi) was examined. When Bi $\geqslant 0.15$, it is impossible to consider the surface temperature as being equal to the temperature at mid-depth. The distribution of water temperatures in depth, at periodical fluctuations of the air temperatures, can be characterized by the Predvoditelev number.

Experimental study

1. An experimental unit was built to study the influence of free-convection heat transfer when vertical and horizontal temperature gradients exist. A series of experiments were carried out in this unit under conditions of vertical temperature gradients. The experimental results were plotted as the function $Nu = f(Gr \times Pr)$ and were compared with previously published data (Figure 238).

2. The test unit has been redesigned for the study of the effect of water-vortex zones on the water stream.

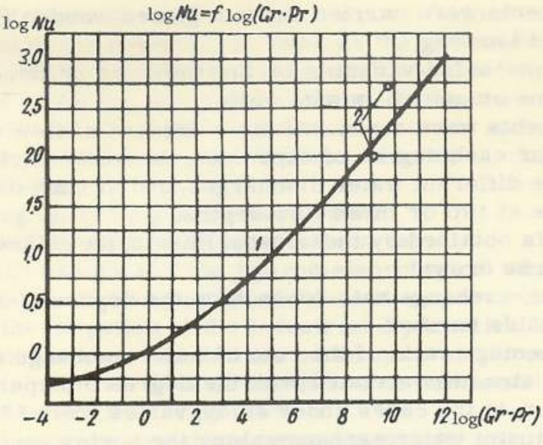


FIGURE 238. Function $Nu = f\,(Gr \times Pr)$.

The second part of the study deals with water discharge between the inflowing warm-water stream and the vortex zones, considering that the stream is calm and steady-flowing.

As the result of interaction between these two hydraulic elements, an exchange of water occurs, the study of which is of significant theoretical and practical interest.

The study of the water exchange is of great importance for the determination of the efficiency of heat exchange in the vortex zones of cooling ponds, and also for clarifying the conditions of silting of storage basins, settling tanks, navigable waterways, and water intakes.

The following studies were performed in 1958:

Experimental methods were devised, installations for the experiments were designed and built, and preliminary experiments were carried out.

Three methods for the experimental establishment of the magnitude of the water exchange were considered: 1) the volumetric method; 2) the electrochemical method; and 3) the statistical method.

In 1958, the water exchange between an inflowing stream and a vortex zone was studied by the statistical method.

The simplest case of liquid flow of a stream with vortex zones has been considered, i.e., the sudden one-sided widening of a stream of a rectangular cross section into a flume with smooth walls and a flat bottom (the two-dimensional problem).

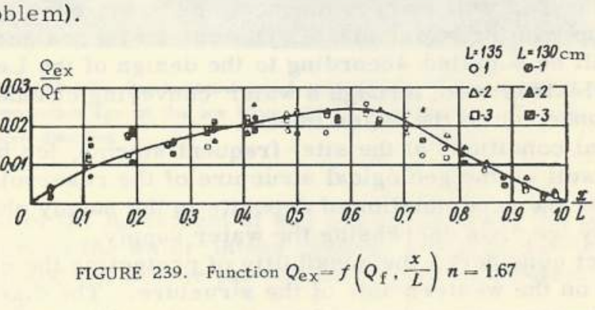


FIGURE 239. Function $Q_{ex} = f\left(Q_f, \frac{x}{L}\right)$ $n = 1.67$

The experiments were carried out in a glazed wooden flume 60 cm wide, 55 cm high, and 14 m long.

The sudden one-sided widening of the flow was produced by a nozzle fixed to the flume on one of its side walls.

The experiments were conducted for 3 degrees of flow expansion: 1.67, 2.5, and 5.0. For each degree of expansion, the water exchange was observed for three different water discharges, and at each discharge, observations were made at two or three flow depths.

From the data obtained by these experiments, the following preliminary conclusions can be drawn:

1. The water-exchange rate depends on the degree of flow widening and on the Reynolds number.

2. The percentage ratio of the rate of water exchange to the discharge of the inflowing stream correlated with the degree of expansion and the Reynolds number, in the cases under study, varies from 13% to 70%.

3. The maximum water exchange along the vortex zone occurs approximately at 2/3 the length of the vortex, measured from the point of sudden flow widening (Figure 239).

The improved methods of hydraulic calculations were adopted by the TUiN for the design of cooling ponds.

HYDRAULIC LABORATORY INVESTIGATIONS OF WATER-INTAKE STRUCTURES OF THE LENENERGO REGIONAL POWER PLANT

Research Team: O. G. Akulova, Engineer
A. N. Demidov, Engineer

The aim of the research performed during the current year at the Industrial Hydraulics and Ice-Thermal Laboratories of the All-Union Scientific Research Institute imeni B. E. Vedeneev, has been the experimental check of the layout of the designed protective structures for the water-intake channels of the Lenenergo Power Plant under construction.

The studies were performed in order to ensure the uninterrupted operation of the intake and also to clarify special hydrological conditions of the area in the autumn and winter seasons, and their influence on the water intake.

The study also considered additional means of protection that might prove necessary, as well as recommendations to increase the reliability of the water supply of the power plant. The water at a designed rate of 64 to 80 m^3/sec will be supplied, according to the design of the Leningrad branch of the Teploelektroproekt, through a water-conveying channel 70 m wide and 618 m long connected to the water intake.

The natural conditions at the site – frequent storms, ice blocking and jamming, as well as the geological structure of the reservoir bottom are liable to cause the accumulation of deposits in the supply channel and its obstruction by ice, thus decreasing the water supply.

The project considered the possibiltiy of protecting the channel by one dike situated on the western side of the structure. The dike is supposed to protect the water-supply channel from the action of the waves, from possible

accumulation of deposits and from other winter factors causing disturbances to the operation of the power plant (such as the movement and forced immersion of the ice due to the wind action, jamming by floating ice blocks that are capable of reducing the cross section of the channel, difficulties caused by frazil ice, which appear when the water is undercooled by winds of long duration, breaking up the ice cover which thus clogs the screens of the water intake).

The following operations were carried out:

1) a survey of the site on the coast of the bay;

2) a study and analysis of the hydrometeorological winter conditions in the region of the bay, based on published data and on previous observations;

3) a model investigation at the Industrial-Hydraulics Laboratory.

The experiments were carried out on an open-air three-dimensional model having an over-all area of 560 m^2. The horizontal scale of the model was 1:250 and the vertical scale 1:50.

Two prevailing wave directions were investigated, from southwest and northwest, with maximum possible wave parameters of $h = 3.0$ m and $\lambda = 50$ m.

The study of the nature of currents forming in the area of the water intake, and of the possible silt deposition caused by these currents, shows that, owing to the absence of suitable protective structures at the supply channel and to the use of a single protective dike, no uninterrupted water supply can be guaranteed. A single dike does not sufficiently protect the channel against ice blocks drifting from the sea due to the action of winds blowing from northwest and northeast.

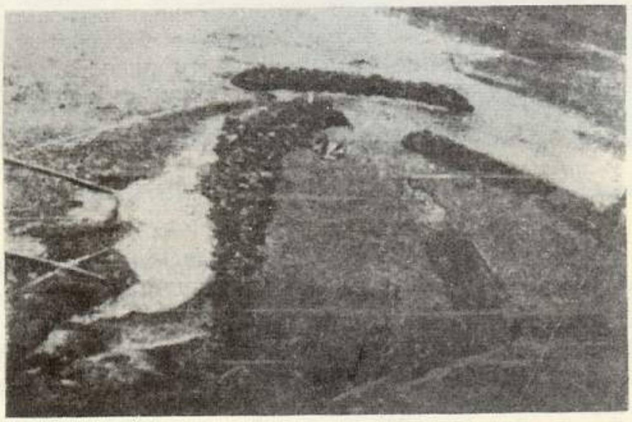


FIGURE 240. Breakwater located perpendicular to the channel for protection against the ice blocks driven by winds from the northwest direction

As a result of the movement of silt occurring along the coastal strip of the bay and across the water-intake channel, silt accumulates in the area of the channel, and the deeper part of the channel is gradually silted up by sand sediments.

The study and comparison of the various alternatives of protective structures showed that the silting zone somewhat shifts from the eastern embankment of the channel toward the shallow shores of the bay, according to the form and position of the structure. The danger of silting of the inlet channel is present in any case.

It has thus become evident that an additional dike is required to protect the channel not only from the east side, but also from the west side.

The eastern dike will stop the advance of the silt toward the channel area and serve as a trap for the accumulating sand outside the forebay of the intake channel.

The same dike will also protect the structures against winds from the northeast direction.

In order to ensure protection against ice blocks driven by winds from the northwest, the construction of a breakwater (Figure 240) would be advisable, situated almost perpendicular to the channel. However, considering the high costs of building a breakwater in deep waters, for protection against the drifting of ice blocks by winds from the northwest direction, the construction of the breakwater may be postponed to a later stage of the expansion of the power plant. Observations through the first years of production will furnish data as to its necessity.

In order to prevent the formation of frazil ice, it is recommended to return the warm waste water into the inlet channel at rates not exceeding $10\,m^3/sec$. The results of the investigations were forwarded to the design agency.

LABORATORY OF INDUSTRIAL HYDRAULICS VNIIG IMENI B. E. VEDENEEV

Head: A. G. Averkiev, Candidate of Technical Sciences, Senior Research Worker

HYDRODYNAMICAL, AERODYNAMICAL AIR THERMAL CONDITIONS IN COOLING TOWERS

Responsible for Research: A. G. Averkiev, Candidate of Technical Sciences, Senior Research Worker

In areas of scarce water sources, the rational design of water-cooling towers becomes most important for ensuring reliable water supply to thermal power stations and industrial plants. Their hydrodynamical, aerodynamical, and thermal behavior should be investigated for this purpose.

The laboratory investigations included the study of cooling towers, with a spray area of 500 to 4000 m^2.

During 1958, investigations were carried out, at the laboratory of industrial hydraulics of VNIIG imeni B. E. Vedeneev, on the aerodynamical operating conditions of a cooling tower with a spray area of 2500 m^2.

The laboratory investigations dealt with the following problems:

1) the study of the behavior of the airflow in the tower, the determination of the distribution of the different airflow velocities on the surface of the water-spray structure, and the location of stagnant or turbulent areas in the spray structure;

2) the determination of the influence on the airflow of:

a) the height of air inlet louvers;

b) the structure (size and shape) of the upper edge of the air-inlet louvers;

c) the type of supporting posts of the tower in the area between the air-inlet louvers;

d) the form of the water-spray system: single-, double-, or triple-stage;

3) the determination of the aerodynamical resistance of the cooler, and the influence of the above factors (Point 2) on the magnitude of this resistance;

4) the establishment of means to avoid the nonuniform distribution of air on the water-spray structure and to reduce the aerodynamical resistance;

5) the study of the aerodynamical conditions in the natural-draft cooling towers subject to the action of wind.

A 1:50 scale wedge-shaped model of a tower was built for aerodynamical studies. The general view of the model is shown in Figure 241.

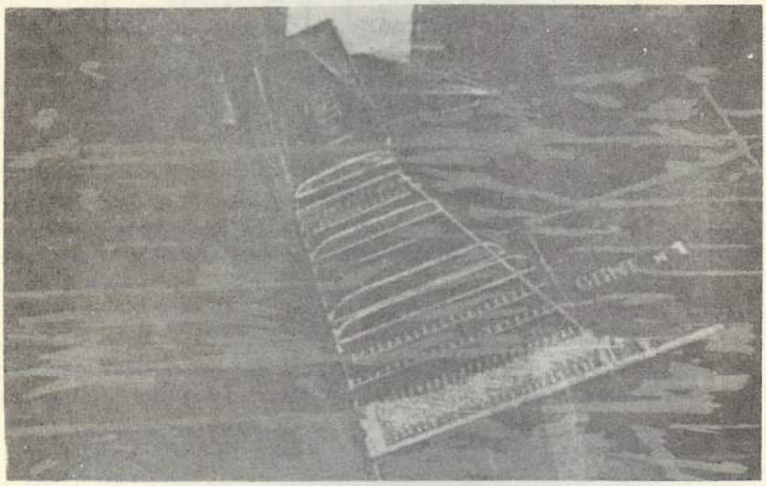


FIGURE 241. General view of the model

The results of the studies on this model led to the following conclusions:

1. The height of the air-inlet louvers has a considerable influence on the magnitude of the aerodynamical resistance of the tower. The optimum height for a 2500 m^2-type tower is to be taken between 5 and 6 m. In this case the over-all resistance of the tower will be $\zeta = 55$ to 65 (Figure 242).

2. The resistance of a cooling tower with a deflector is smaller than that of one without a deflector. The most efficient deflector has a length $l = 1.5$ m and an inclination to the horizontal plane of 5°.

3. By trying out various types of columns, it was observed that the column cross sections have no considerable influence on the over-all resistance of the tower. When using a 1.5 m long deflector, it is recommended not to support it by posts, but by cantilevers or supporting rods.

4. Experiments to determine the negative influence of strong winds on the operation of towers were carried out on a special unit inside a hydraulic flume. These experiments showed that a few engineering measures can correct the internal pattern of the flow during periods of strong wind. Such

measures are, e.g., opening and closing the air-inlet louvers on the sea side of the cooling tower, and the removal of the wind partition inside the tower.

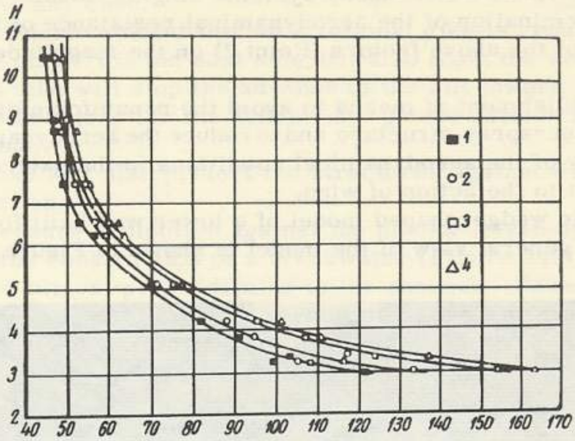


FIGURE 242. Function $\zeta_{total} = f(H)$

1 - with deflector, without columns; 2 - without deflector, without columns; 3 - with deflector, with columns; 4 - without deflector with columns.

5. In addition to the aerodynamical study, the laboratory equipment was redesigned and experimental units were prepared for thermal investigations.

HYDRAULIC LABORATORY INVESTIGATIONS OF THE WATER-INTAKE STRUCTURES OF THE TOM'-USINSK REGIONAL ELECTRIC POWER PLANT

Responsible for Research: G. N. Lapshin, Candidate of Technical Sciences, Senior Research Worker

The Leningrad branch of Teploelektroproekt designed a scheme for industrial water supply to the Tom'-Usinsk power plant, which considered the establishment of two water-intake structures, situated on a 3 km stretch of the Tom' River.

The hydraulic laboratory investigations consisted of: a) a laboratory testing of the water-intake operation under summer and winter conditions; b) the determination of the amount of water drawn from the Tom' River by the combined operation of both intake structures; c) checking of the suitability of the location and of the shape chosen for the second water intake; d) the establishment of means to improve conditions of operation, etc.

The laboratory experiments were carried out on two three-dimensional models: a 1:1000 scale hydrodynamical model (Figure 243) for a stretch of the Tom' River including both water intakes, and a hydraulic open-air

1:120 scale model for a stretch of the Tom' River with intake No. 2 only (Figure 244).

On the aerodynamical model, general hydraulical problems were studied: the relative water discharges from the river into both inlets, the distribution of the water discharge between the channels, and the location of the warm waste-water discharge.

On the open-air model, the shape of intake No. 2 was determined so as to improve its efficiency.

The results of the laboratory experiments led to the following conclusions.

1. During the summer, when the water flow in the river is clean, a confidence factor of 97% can be reckoned on for an inflow discharge of 65 to 70 m^3/sec from the river, if both intakes are constructed according to the design, and the Volodina Channel is being utilized. The same factor can be assumed for discharge of 20 m^3/sec in winter.

2. The analysis of hydrological data, field observations, and laboratory investigations established that the operating conditions of intake No. 1 during winter, and especially during ice freezing and ice drift, are very severe. Therefore, in order to ensure uninterrupted water supply to the Tom'-Usinsk power plant, the provision of two water intakes is imperative. One of the intakes will utilize the Volodina Channel, and the other will be located 3 km upstream at the beginning of the Kol'chezasskii shallow.

Laboratory studies of water intake No. 2, with different locations and shapes of the headworks showed that, under the prevailing conditions, it is essential to construct an intake with bottom feeding. Investigations of the bottom-feeding intake showed that there is no danger of the headworks being blocked by ice. The design requirements do not provide therefore a waste-water discharge near intake No. 2.

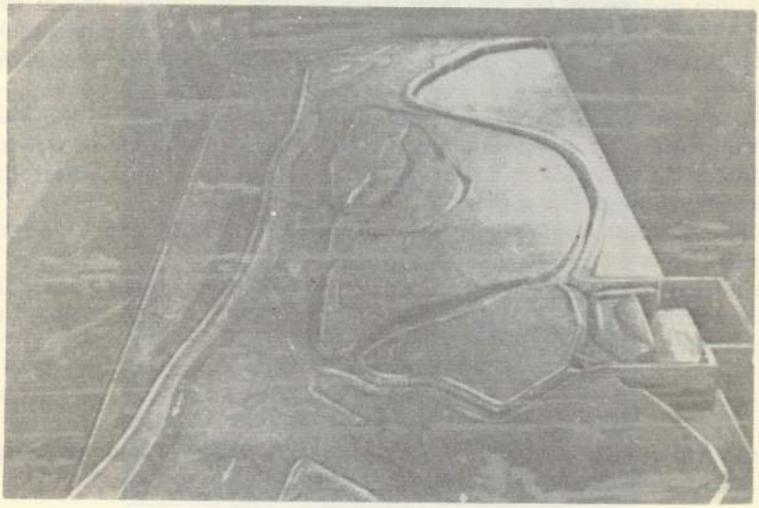


FIGURE 243. Aerodynamical model

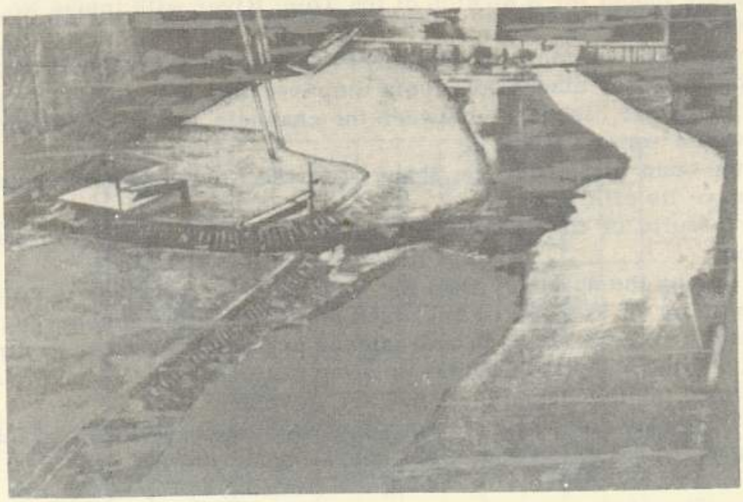


FIGURE 244. Open-air hydraulic model

3. In order to protect the channel from sedimentation and accumulation of silt, dredging equipment for periodical cleaning of the channel should be provided. The construction of a water-deflecting dike with its crest somewhat above the maximum water level during floating-ice drift should not be adopted, as it may cause undesirable deformations in the main river bed.

4. In order to protect intake No. 1 against the severe winter conditions under which it will have to operate, a waste-water discharge will have to be provided upstream from intake No. 1 in addition to the one at the headworks of the Volodina Channel. The discharge of this channel according to the calculations by the Ice Thermal Laboratory of VNIIG is approximately $10\,m^3/sec$.

5. Considering the frequent formation of floating ice in the shallow, the warm-water discharge should be located at a distance of 0.6 to 0.7 km from intake No. 1. Investigations of location of the warm-water discharge in the river showed satisfactory results.

6. The discharge of warm water downstream from the Volodina Channel (intake No. 1) is of little use.

The result of the study in the form of a technical report and conclusions were forwarded to the Leningrad Branch of "Teploelektroproekt" for use in their technical designs.

AERODYNAMIC LABORATORY RESEARCH ON FORCED-DRAFT MULTI-UNIT COOLING TOWERS OF THE POWER PLANT AT KEIVELI (INDIA)

Responsible for Research: L. E. Rode, Junior Research Worker

Investigations were carried out on a multi-unit, double-flow, forced-draft cooling tower (Figure 245) to develop the most rational outline of the units

as well as the shape and dimensions of the component elements of the tower that will ensure the best aerodynamical conditions and hence the most efficient cooling of the circulation water of thermal power plants.

The studies were carried out on two- and three-dimensional models with transparent panels built to a scale of 1:20 (Figure 246).

As a result of the investigations, the value of the resistance factor was obtained for three layouts of the cooling tower and a diagram prepared showing the distribution of the air velocities above the spray system.

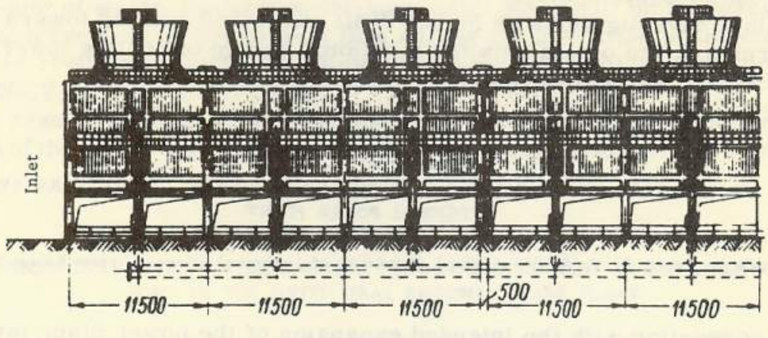


FIGURE 245. View of a double-flow, forced-draft, multi-unit cooling tower

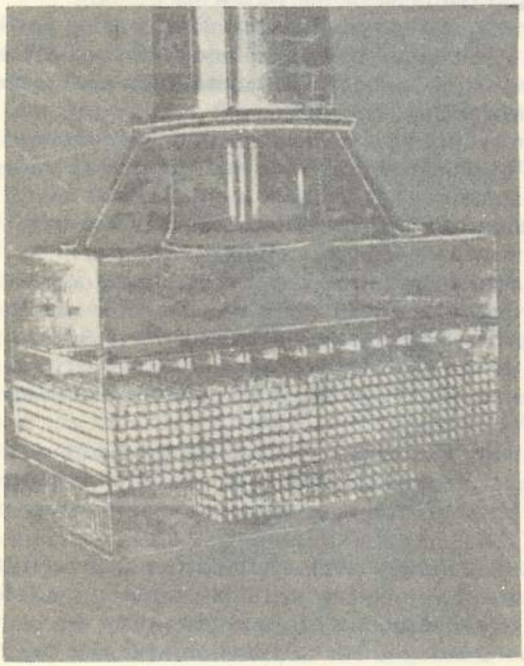


FIGURE 246. Model of cooling tower

The technical report of the study was forwarded to the Leningrad Branch of the "Teploelektroproekt" Institute for use in cooling-tower designs.

The investigations originally intended for a specific project have produced data which can be used in the future design of multi-unit forced-draft cooling towers. It will enable a more rational approach to the selection of proper dimensions of the component forces, and to determining the position of the fans. The investigations also showed the influence of the different cooling elements on the value of the aerodynamical resistance, i. e., height of louvers, length and form of inlet deflectors, and number of stages of the spray system, etc.

Future investigations on forced-draft multi-unit cooling towers should be carried out to obtain data for a rational design of cooling towers of a small aerodynamical resistance.

LABORATORY INVESTIGATIONS OF A COOLING POND FOR THE BELOYARSKAYA REGIONAL POWER PLANT

Responsible for Research: G. V. Vostrzhel, Candidate of Technical Sciences, Senior Research Worker

In connection with the intended expansion of the power plant, investigations were conducted for more efficient use of the pond for the cooling of the inflowing warm-water stream.

The laboratory was confronted with the problem of finding the best scheme for the outlet into the pond of the inflowing stream.

The hydraulic study of the stream in the cooling pond of the Beloyarskaya power plant was conducted by the method of forced discharge into an outdoor three-dimensional model built to a horizontal scale of 1:700 and a vertical scale of 1:150.

Six different systems were investigated, all developed by the Leningrad Branch of the "Teploelektroproekt" Institute which provided the designs of two discharge structures with their technological specifications.

From the study of the form of spreading of the inflowing stream it was found that:

a) the increase of the inflow discharge can be achieved, when the stream is released through end outlet No. 1 and the inlet into the pond is widened up to 200 m. Increasing the width beyond 200 m does not produce the desired increase in the discharge of the inflowing stream;

b) the downstream part of the pond can be utilized by the combined operation of both water outlets Nos 1 and 2.

Water outlet No. 1 borders on the site of the power plant and evidently requires the least capital investments. Special attention has therefore been given to its maximum operation in the first stage of the plant's operation.

With this aim in view, a series of additional investigations were carried out, which showed that locating outlet No. 1 along the left-hand bank toward the earth dike is undesirable. In this case only the central part of the pond's waters are being used. Operation of outlet No. 2 in addition to outlet No. 1 permits the use of the downstream part of the pond.

In order to improve the efficiency in using the pond, the laboratory proposed a new design for the water-discharge structures. According to this

design, outlet No. 1 will be formed as a 200 m wide stream-deflecting dike, discharging the stream toward the left-bank cape. In this case the upstream part of the pond can be effectively used, as the size of the inflowing stream is considerably larger than in the former designs. Operation of the outlet No. 2 in this case enables the use of the waters of the entire reservoir, i. e., all three parts (downstream, central, and upstream).

The proposed design was checked by passing a warm-water stream through the model.

It is evident that at the first period of operation of the power plant the use of outlet No. 1 will be sufficient. With the expansion of the power plant, the amount of water discharged through outlet No. 1 will be increased, too. Field observations of the natural flow during the operation of the cooling pond are required in order to establish the maximum discharge through outlet No. 2.

The results of these investigations were handed in to the Leningrad Branch of the "Teploelektroproekt" with the purpose of improving future design.

LABORATORY INVESTIGATIONS OF THE STREAMFLOW PATTERN IN THE COOLING POND OF THE SOUTH URAL REGIONAL POWER PLANT

Responsible for Research: G. V Vostrzhel, Candidate of Technical Sciences, Senior Research Worker

In connection with the expansion of the power plant the laboratory of industrial hydraulics of VNIIG carried out hydraulic investigations of the flow pattern in the cooling pond with the aim of improving the efficiency of the existing cooling surface of the pond.

Calculations have shown that when the output of the plants will be increased up to the designed value, the water level in the pond will drop. Lowering the water level below its normal value will result in the formation of a sand bank on the right-hand shore of the pond, with a shallow water region near it.

Therefore, the laboratory investigations of the streamflow hydraulics in the pond had to determine the engineering measures necessary to ensure the increase of the cooling-pond area. The design included the dredging of the bottom in order to remove the sand bank on the right-hand shore.

The volume of dredging work and the outline of the area to be dredged should be determined from hydraulic investigations of the flow lines on the model of the South Ural Regional power plant pond.

For carrying out the laboratory study, an aerodynamic three-dimensional model was constructed to a horizontal scale of 1:1000 and a vertical scale of 1:200. The free surface of the pond was simulated by glass panes.

The propagation of the airflow was investigated by coloring the air with smoke, whose movement enabled the establishment of the boundaries of the inflowing water stream and the location of the vortex zones.

The velocity of the air stream was measured on the vertical at a depth corresponding in nature to 1 m below the free water surface.

As a result of the laboratory studies of the spreading of the transitory inflowing stream in the water of the cooling pond of the South Ural Regional power plant, the following conclusions can be reached:

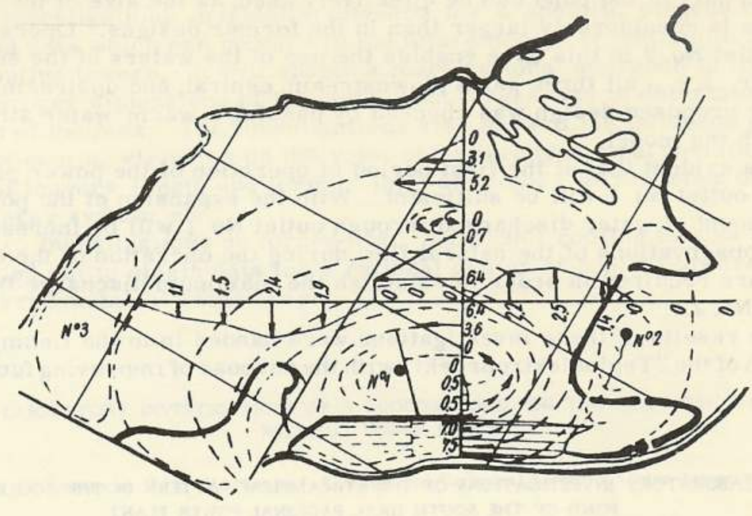


FIGURE 247. Contours of vortex and still-water zones

1. The drop of the water level in the cooling pond due to the increased output of the power plant creates unfavorable hydraulic conditions for the spreading of the inflowing stream in the water of the pond. The study showed that approximately one half of the area of circulation is occupied by vortex and still-water zones (Figure 247).

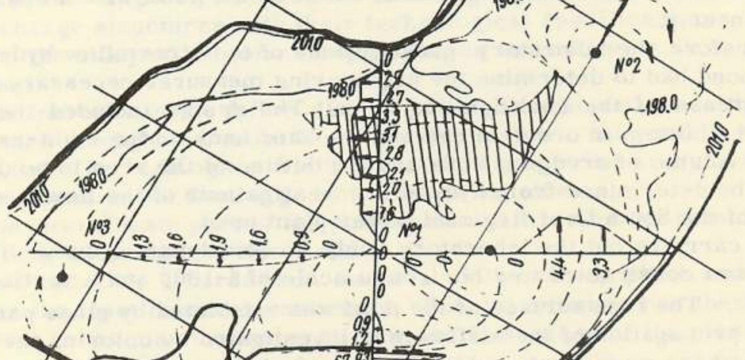


FIGURE 248. Contours of recommended dredging of the sand banks on the right-hand shore

2. An increase in the efficiency of the cooling pond of the South Ural Regional power plant can be achieved by dredging the bottom of the pond in order to remove the sand banks on the right-hand shore. The contours of the recommended dredging works are shown in Figure 248. The investigations showed that by these engineering measures (dredging, grading, etc.) it is possible to obtain a satisfactory flow pattern of the inflowing stream, and no other additional operations, such as the dredging of the left-hand shore of the inlet stretch at the bend of the Uvel'ka River, or the lengthening of the stream-deflecting dike, are required.

3. Investigations showed that the dredging of the sand bank on the right-hand shore has no influence whatsoever on the hydraulics of the stream when the water level is raised.

The investigations carried out were forwarded to the Leningrad Branch of Teploelektroproekt for use in the design of output extension of the South Ural Regional power plant.

COMPILATION OF THE SECOND EDITION OF ENGINEERING SPECIFICATIONS FOR THE DESIGN OF COOLING PONDS

Responsible for Research: E. V. Vostrzhel, Candidate of Technical Sciences, Senior Research Worker

The first edition of the proposed Engineering Specifications for the design of cooling-water ponds, compiled in 1957 in connection with the study "Completion of experimental work and preparation of standards for the design of cooling ponds", has been extensively revised. As a result, a second edition of Engineering Specifications was prepared. Representatives of the Leningrad Branch of Teploelektroproekt (B. S. Fedorovski, V. E. Andrianov, and M. L. Prutsyn) have participated together with the VNIIG personnel in the preparation of the Engineering Specifications.

The draft for the revised edition of the Engineering Specifications for the design of cooling ponds consists of the following chapters:

1. General propositions including the limits of application of the Specifications, terminology, and basic design data.
2. Schemes for the use of cooling ponds.
3. Hydraulic design of cooling ponds.
4. Meteorological data for the design of cooling ponds.
5. Thermal design of cooling ponds.
6. Determination of the magnitude of surface evaporation in cooling ponds.

The appendixes to the Engineering Specifications give examples of design for the last three chapters of the Specifications.

The draft of the Specifications for the design of cooling ponds is discussed by the Scientific Council of the Institute.

LABORATORY INVESTIGATIONS OF AIR ENTRAINMENT INTO THE FLOCCULATION TANKS OF CLARIFIERS

Responsible for Research: L. E. Rode, Junior Research Worker

The investigations were conducted to find a method for increasing the amount of air entrained into the flocculation tank by the sewage stream. The experiments were carried out on the hydraulic installations of the laboratory reproducing on a natural scale the feeding flume connected to the clarifier upright pipe (Figure 249). The difference in levels between the feeding flume and the clarifier was used for air entrainment. The amount of entrained air was measured by gas-flow-rate meters attached to a pipe connection in the airtight cover of the flocculation tanks.

Various designs of the inlet section of the uprise were investigated with the aim of intensifying the air entrainment for a given level difference of 0.6 to 1.0 m.

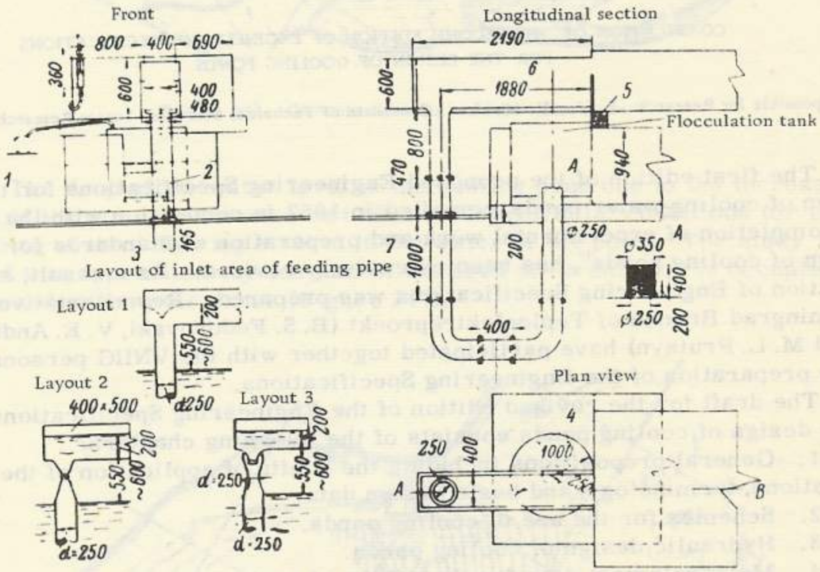


FIGURE 249. Laboratory unit for investigation of air entrainment into the flocculation tank of a clarifier

1 - gas-flow-rate meters; 2 - inlet stretch made of plexiglass pipe; 3 - cast-iron pipe; 4 - pipe connections for air outlet; 5 - measuring weir; 6 - feeding flume; 7 - feeding pipe.

The best air-entrainment results were obtained in a design with a straight stretch of inlet pipe at a distance of 2.0 m from the measuring weir in the feeding flume.

Figure 250 shows variation of the percentage of entrained air with the water discharge.

A report on the results of the investigation was forwarded to the Leningrad Civil Engineering Institute, Department of Canalization, which is compiling instructions for improvements in the operation of settling tanks.

The study provided a clear answer to the problem of calculating the amount of air entrainment (without artificial aeriation) into the flocculation tank, as a percentage of the sewage discharge into the tank.

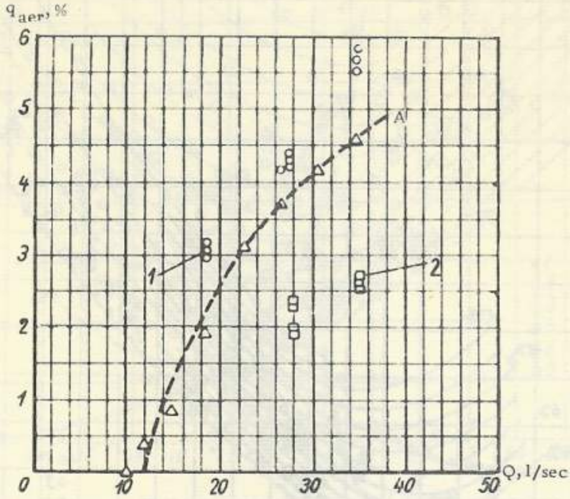


FIGURE 250. Curve of variation of air entrainment with the percentage of sewage discharge in l/sec (for a straight feeding pipe)

A - Curve $q(Q)$ for a rectangular flume end; 1 - measuring points for air inlet after redesigning the flume end to suit natural conditions, with narrowing of the pipe entrance; 2 - measuring points for air inlet, with the installation of a measuring weir at a distance of 80 cm from the flume end.

HYDROMECHANIZATION LABORATORY OF THE VNIIG IMENI B. E. VEDENEEV

Head: Professor M. A. Dement'ev, Doctor of Technical Sciences

STUDIES OF THE METHODS FOR DESIGN AND CONSTRUCTION OF HYDRAULIC ASH-AND-SLAG-REMOVAL SYSTEMS

Responsible for Research: S. I. Goryunov, Candidate of Technical Sciences, Senior Research Worker

The ultimate aim of the investigation was the establishment of methods for the design of hydraulic ash-and-slag-removal systems for thermal power plants.

During the last year, the field-size hydraulic ash-conveying unit of the laboratory has been redesigned and markedly improved.

1. The size of the unit has been more than doubled, which ensured more accurate observation. At the same time, the length of pipelines was increased to 80 m.

2. A continuous uniform feeding of large lumps of slag to the pipeline over a wide range of conveying rates was ensured.

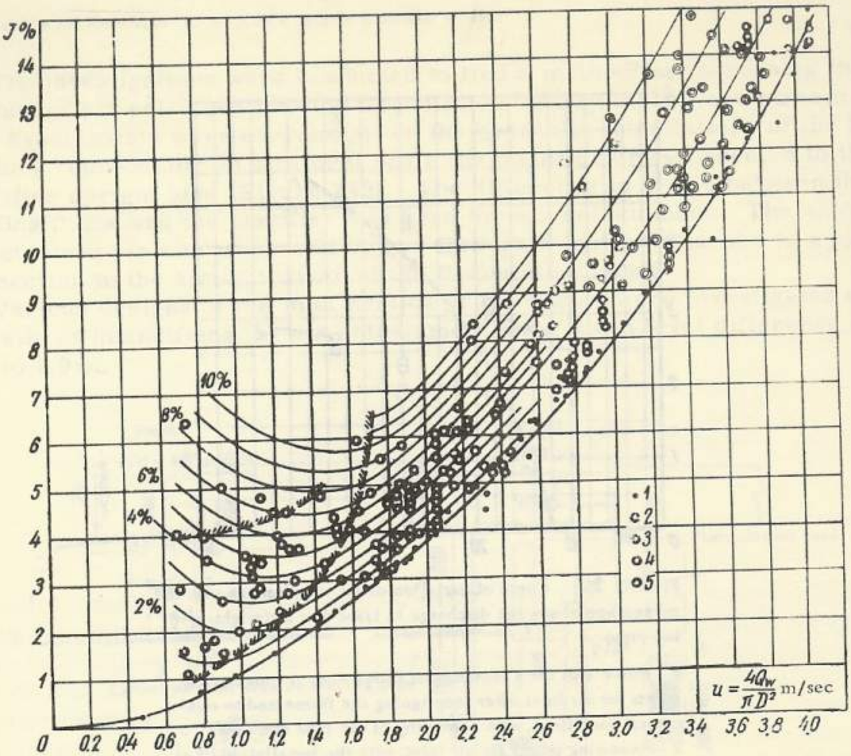


FIGURE 251. Head losses during the hydraulic conveying of slag through a glass pipe with D = 75 mm

1 - pure water; 2 - water + slag, flow with no particle settling; 3 - water + slag, sediments accumulating on the bottom in the form of ridges; 4 - water + [slag] ridges; 5 - water + slag, settling in a continuous layer.

3. Conditions for the hydraulic conveying of solid particles have been created with an addition of air in an amount ranging from 0 to 150%.

4. The construction of the pressure valves and of other elements of the unit has been improved.

The study showed the possibility of performing part of the experiments with the unit operating in closed circuit, and the limits of applicability of closed-circuit operation were established.

Over 300 experiments for the investigation of hydraulic slag removal were conducted during the summer and the autumn.

Experiments were carried out with various additional quantities of air and without any additional air, as well as with different slag contents varying from 1 to 15%.

Investigations were made of the conditions of hydro transportation in glass pipelines of 75 mm and 100 mm diameters. Observations were made of the state of flow and movement of solid materials.

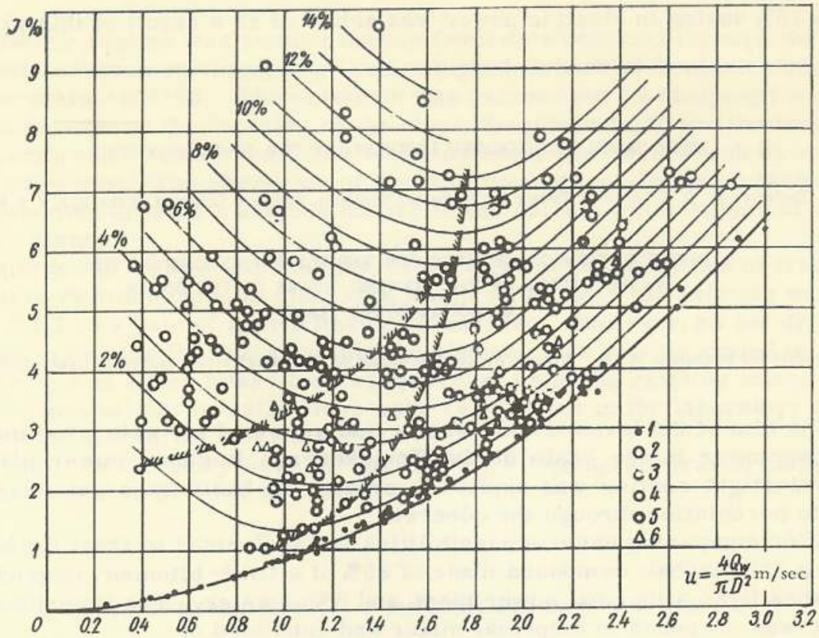


FIGURE 252. Head losses in the hydro transportation of slag in a glass pipe of D = 100 mm

1 - clean water; 2 - water + slag, with no sedimentation; 3 - water + slag, sediments accumulating in ridges on the bottom; 4 - water + slag, ridges; 5 - water + slag, complete sedimentation; 6 - water + slag, experiments of 1957 (c = 3-5%)

Two diagrams were drawn from the data obtained during the experiments, showing the dependence of head losses on the velocity of transportation, for the above (1 - 15%) slag contents (Figures 251 and 252).

Experiments have shown that by decreasing the air contents in the system, the zone of its positive influence on head losses is moved toward the higher velocities, while its absolute value decreases at the same time.

TESTING THE OPERATION OF THE HYDRAULIC SLAG-REMOVAL SYSTEM FOR FOUR OPERATING BOILERS

Responsible for Research: E. Z. Nagli, Senior Engineer

The aim of the present investigation includes:

1) the proper dimensioning of hydraulic slag-removal systems for operating boilers;

2) the establishment of maximum sludge-pump delivery;

3) the establishment of the hydraulic resistance and the required slurry velocity in the ash pipes.

As a result of the investigation, the dimensions of the hydraulic system were established so as to guarantee its undisturbed operation. The most economical conditions for the system were obtained by reducing the pressure and the discharge of the ejecting water.

A 25% saving in electric power was achieved as a result of this work.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR WATERPROOFING

Head: Professor P D Glebov, Doctor of Technical Sciences, Honored Scientist of the R. S. F. S. R.

INVESTIGATION OF METHODS FOR THE WATERPROOFING OF THE WATER INTAKE OF THE COOLING TOWER OF THE TOMSK REGIONAL POWER PLANT

Responsible for Research: S. N. Popchenko, Candidate of Technical Sciences, Senior Research Worker

The aim of the investigation was to find a method for waterproofing the cooling-tower intake basin of the Tomskenergo Regional power plant.

Watertight coating was applied to protect the basin from water losses due to percolation through the concrete.

After comparing various possibilities it was decided to treat the basin with a cold asphalt compound made of 85% of a lime-bitumen compound, 10% of ashes from a thermal power plant, and 5% of water. The lime-bituminous paste was prepared in a special mixer and consisted of:

BN-Sh Bitumen	55.2%
Lime, first-grade quality	12.3%
Water	32.5%

The output capacity of the paste mixer reached 2.8 tons per shift. The cold asphalt compound was applied to the horizontal surface by pouring followed by smoothing (two layers with a total thickness up to 30 mm were applied); its application on the walls was achieved with the aid of a VNIIG-4 asphalt spray gun in three sprays of a total layer thickness of 15 mm. The whole sealing operation of the basin having an area of about 2000 m^2 lasted 45 days. The work was done in a traveling cabin heated by steam radiators. Trial filling of the basin proved the satisfactory quality of the sealing.

ICE-THERMAL LABORATORY OF THE VNIIG IMENI B. E. VEDENEEV

Head: Professor A. M Estifeev

LABORATORY INVESTIGATIONS OF THE WATER-INTAKE STRUCTURES OF THE TOM'-USINSK POWER PLANT

Responsible for Research: G S. Shadrin, Candidate of Technical Sciences, Senior Research Worker

The aim of the investigation was the assessment of the ice regime in the region of the water intake of the power plant and the recommendation of means to overcome possible ice troubles.

The ice regime was studied mainly from data obtained through the hydrological research performed by the Leningrad Branch of Teploelektroproekt in the years 1950-56. The research was carried out by studying the relationship between the freezing of the river, the river runoff, and the drop of air temperatures. It was found that ice jams associated with ice drift occur almost yearly. The character of ice drift depends on the above factors. Ice jamming creates a serious danger to the normal water supply of the power plant.

In order to reduce ice troubles, especially during formation of frazil ice, it is recommended to discharge warm water near the existing water intake No. 1 to a rate of approximately 10 m^3/sec. However, as ice drift may temporarily clog the entrance into the channel, it is recommended to provide a second water intake located upstream from the existing intake No. 1.

The hydraulical investigations were carried out in the laboratory for industrial hydraulics of VNIIG.

Results of the study were forwarded to the Leningrad Branch of Teploelektroproekt to be used in design.

CHAIR OF HYDROTECHNICAL STRUCTURES MISI IMENI V. V. KUIBYSHEV

Head: Professor M. M. Grishin, Honored Scientist of the R. S. F. S. R., Member of the Academy of Construction and Architecture

HYDROTECHNICAL STRUCTURES OF FISH-BREEDING PONDS. HYDROLOGICAL INVESTIGATIONS OF WATER OUTLETS FROM THE FISH POND

Scientific Guidance: N. P. Rozanov, Candidate of Technical Sciences, Lecturer

Head of Research Team: A. I. Antipov, Candidate of Technical Sciences, Senior Research Worker

Research Team: V. V. Korneeva, Engineer
O. V. Evtushenko, Technician

The aim of the study was to check the operation of water-outlet structures of fish-breeding ponds designed by the Gidrorybproekt, and to introduce modifications (based on laboratory experiments) which will improve the operation of these structures.

The designed water-discharge structure consists of a 0.482 m diameter pipe, 20 m long, embedded in the earth dike surrounding the pond. At its upstream side the discharge pipe is provided with gate valves. Its inlet section is protected by special screens. The screens in front of the water-discharge pipes are either No. 4 woven screens, or screens punched with 4 mm diameter holes. A 60 cm deep inlet channel leads to the pipe.

The downstream end of the pipe is provided with an overfall, at the end of which stoplogs can be inserted. A channel leads the water from the overfall to the main collector.

The maximum head at the water outlet is 3 m. Fish pass through the channel at a head of 1.0 m. The maximum permissible inflow velocity at the headwater screens is 10 to 15 cm/sec.

The investigations carried out included: 1) the determination of the discharge capacity of the outlet, and the improvement of the methods of its

hydraulic analysis; 2) the determination of the velocity of approach to the screens for heads of 3.0, 2.0, and 1.0 m, and for screen structures; 3) the determination of the discharge capacity of the water outlet at different degrees of gate-valve openings; 4) the determination of flow velocities at the outflow channel, in order to establish the type and extent of the protective structures; 5) the design of simplest means for dissipation of the energy of the discharged flow; 6) the introduction of constructional modifications in the design and their checking on a model.

The study was performed on a model built to a scale of 1:7.18.

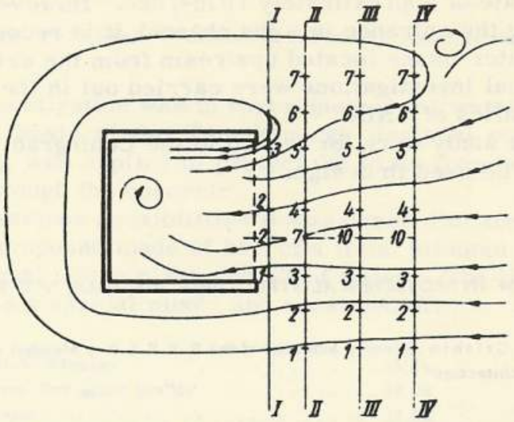


FIGURE 253. Boundary of transient-flow conditions in the pipe (submerged and nonsubmerged)

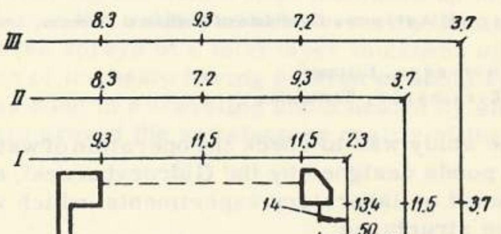


FIGURE 254. Discharge capacity of the structure at different gate-valve openings

The following results were obtained from this investigation.

1. The boundaries of the transient-flow conditions in the pipe, submerged and nonsubmerged, were determined and methods for analysis of these conditions established.
2. The discharge-capacity of the outlet structures for different gate-valve openings were determined, and the resistance factor at the inlet, along the pipe, and at its outlet were established.
3. The velocity distribution at the headwater area of the pipe and the velocity diagrams at the tailwater area were calculated for various heads and structures of the pipe headwork (Figures 253, 254, 255, 256).

The typical design of the headwork structure is not suitable because of the inadmissibly high velocities at the screens.

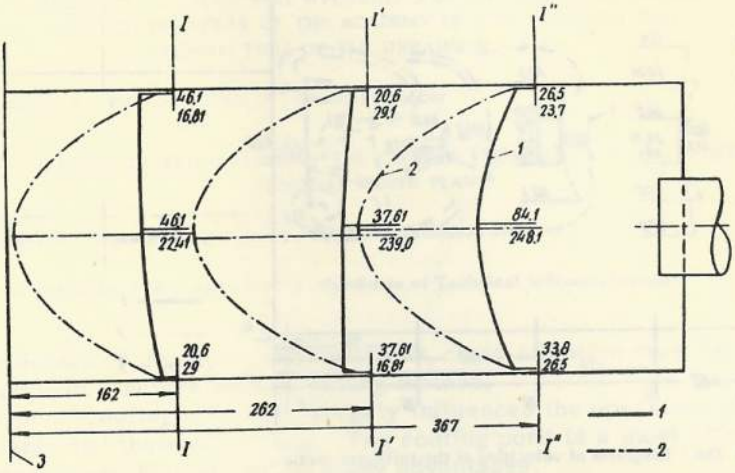


FIGURE 255. Diagram of velocities at the overfall

1 - velocities of bottom currents; 2 - velocities at the water surface.

The laboratory data on the velocity distribution permits the determination of the required modifications for the headwork design and for the shape of the inlet channel. As a result, the inflow velocities may be considerably reduced, the pattern flow improved, and the velocities at different depths equalized, thus obtaining a sufficiently uniform flow of water into the headwork of the pipe.

In order to avoid formation of whirls which are particularly dangerous at a head of 1.0 m when fish are let through, installation of a deflecting nose near the wall of the headwork is recommended. The dimensions of the nose are indicated.

For efficient dissipation of the stream energy at the tailwater area of the pipe, it is recommended, based on the study of several variants, to provide three slotted stoplogs. This will considerably reduce the velocity of approach to the outlet channel and thus reduce the dimensions of channel protective structures for the whole range of possible variations in the head. The provision of such stoplogs creates favorable conditions for fishing along the overfall.

The following was established from the investigation of the bottom-outlet operation:

1. The water outlet operates at its tailwater section under conditions of slight submersion. The discharge coefficient $\mu_{tail.av}$ of the water outlet with its head, but without upright pipe and with No. 4 screen is 0.64; with perforated sheet, $\mu_{tail.av} = 0.63$; with upright pipe and perforated sheet, $\mu_{tail.av} = 0.62$. For nonsubmerged conditions without upright pipe, but with perforated sheet, $\mu_{tail.av} = 0.62$.

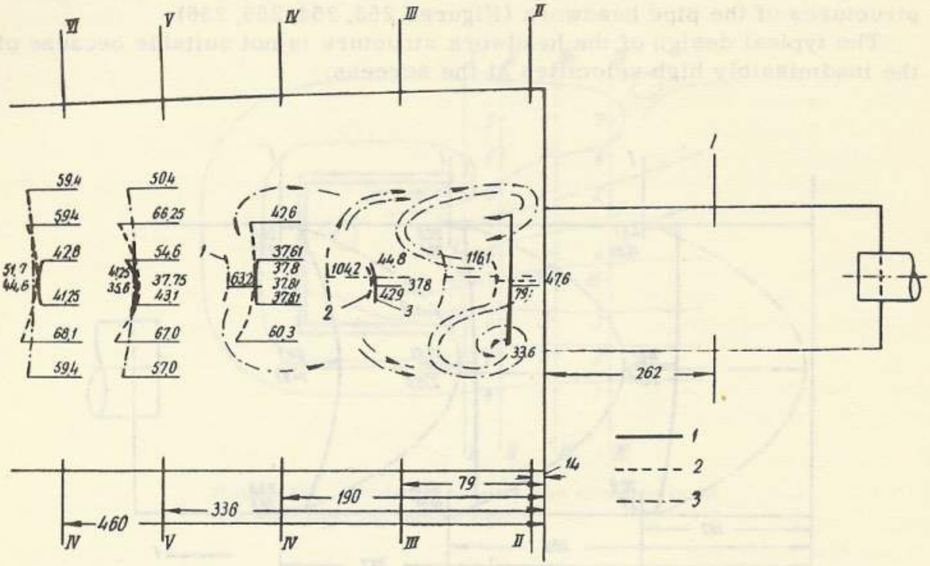


FIGURE 256. Diagrams of velocities at the tailwater section

1 - diagrams of bottom velocities; 2 - diagrams of midstream velocities; 3 - diagrams of velocity at the water surface.

2. Variation of the discharge capacity of the pipe with the opening angle of the valve.

3. In the water outlet with No. 4 screen (no upright pipe used) the velocity of approach to the screens increases with the increase in depth of the pond, reaching a maximum value for $H = 1.0$ m.

With the installation of a perforated sheet (mesh openings up to 4 mm) the velocity of the inflow stream becomes more uniform.

To suit the admissible approach velocities, the screen structure should be widened to 1.5 m and increased in length to 1.0 to 1.2 m.

4. For a water outlet with upright pipe, the velocity of approach is twice as high as in an outlet without upright pipes. The sizes of screens to be installed at the upright pipe should be (2.8 to 3.0)×(2.8 to 3.0) m.

5. To ensure a more uniform velocity of approach to the screen it is necessary:

a) to widen the channel opening at both sides of its headworks by 1.3 to 1.5 m;

b) to remove the wall sustaining the lateral walls and the stoplogs.

6. A diaphragm should be installed at the rear wall which will eliminate formation of water funnels below the pipe.

7. It is also recommended to install three slotted stoplogs as energy dissipators.

previous years and its experience has been used in the present investigation. However, new experience was gathered as to the methods for investigating and calculating such structures.

The results of hydraulic calculations from this study fully confirm the results obtained earlier by the Department,thus permitting their recommendation for more general use.

LABORATORY OF INDUSTRIAL HYDRAULICS OF THE VODGEO SCIENTIFIC RESEARCH INSTITUTE OF THE ACADEMY OF CONSTRUCTION AND ARCHITECTURE OF THE UKRAINIAN S. S. R.

Head: Professor G. A. Petrov, Doctor of Technical Sciences

LABORATORY INVESTIGATIONS OF THE COOLING STRUCTURES OF THE ZMIEV REGIONAL POWER PLANT

Scientific Guidance: Professor G. A. Petrov, Doctor of Technical Sciences

Responsible for Research: I. A. Zababurin, Candidate of Technical Sciences, Lecturer

In the design of thermal power stations, close attention must be given to the selection of a return water-cooling method.

The type of cooling structure largely influences the operation of thermal power plants and their efficiency. The cooling pond is a most efficient type, and is widely used because of its many advantages.

For the Zmiev Regional Power Plant it is proposed to use Lake Liman as a water-supply source, as well as a cooling reservoir.

Since the problem could not be solved theoretically, it became necessary to carry out laboratory research.

This reservoir has an almost circular shape in plan and an average depth of 4.5 m and the aim of the design work was to make the best use of the existing facilities.

The experiments were performed on two models of equal horizontal scale but of two different vertical scales (1:4 and 1:20). The area of the model was approximately 80 m^2.

In addition, the laboratory devised a novel method for closed-cycle operation of cooling ponds. In this method the waste warm water is removed either through surface outlets at certain points on the circumference of the reservoir or through bottom outlets (Figure 257) where the water is fed to the headwork through pressure conduits laid on the bottom so as to avoid obstruction of the water inflow to the pond.

A characteristic feature of this method of utilizing the cooling reservoir is the arrangement of the water intake in the center of the reservoir. A circular, fully submerged, bottom intake was adopted. Water is fed from the intake to the pumping station through a special water conduit.

The following characteristics of the water flow in the new operation schedule methods were observed and established by the laboratory investigations:

1. The entire amount of water in the reservoir is subject to a rotational movement, by which the streams from the outlet points move toward the intake in spiral trajectories. In this way the water particles must travel a considerable distance before reaching the intake, which enhances the cooling properties of the pond. The main reason for the rotary movement of the water mass is formation of a vortex near the intake, where the water along the intake structure reaches high velocities.

2. The distribution of bottom and surface velocities is of great practical importance. For instance, the considerable velocities observed along the shoreline enable the use of the shallower zones, which are not used in ordinary cooling ponds. The velocity distribution is characterized by the almost uniform velocities at the surface and at the bottom, which creates increased turbulence and permits the use of larger volume of the reservoir for cooling purposes.

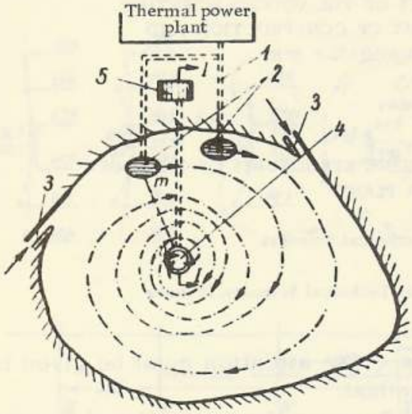


FIGURE 257. Layout of outlet structures for warm waste water

1 - pressure pipe lines for waste-water outlet; 2 - bottom outlet heads; 3 - surface water outlet; 4 - ring-shaped bottom outlet; 5 - pumping station.

The following data were also established through these investigations:

a) the position of the central intake;

b) the influence of the intake head structure on the hydraulical operating conditions of the cooler;

c) the influence of the direction of outlet mouths on the velocity conditions in the reservoir;

d) the influence of the discharge velocities on the hydraulic conditions of the reservoir;

e) the influence of waves on the stream distribution.

The data obtained improved the design of all the structures of the circulation-water supply system for thermal power stations with water reservoirs of an almost circular form.

It should be noted that the use of this method requires the emptying of the reservoir for the construction of the central water intake.

The above data have been used by the Khar'kov Branch of Teploelektroproekt for the design and construction of the Zmiev Regional Power Plant.

DEPARTMENT FOR HYDROELECTRIC AND PUMPING STATIONS OF THE NIMI

LABORATORY INVESTIGATIONS OF THE COOLING POND AT THE KRASNODAR THERMAL POWER PLANT

Responsible for Research: V. I. Petrov, Candidate of Technical Sciences, Lecturer

Research by: V. A. Orusskii, Candidate of Technical Sciences, Lecturer

1. As is known from the practice of cooling-pond operation, the stream of warm water flowing from its outlet into the pond toward the intake occupies only a small part of the pond surface. The rest of the pond surface, sometimes considerably large, consists of stagnant and vortex zones, taking no part in the advance of circulation water, thus reducing its cooling effect. The longer the distance and the time it takes the stream to travel on its cooling course, and the larger the evaporation surface of the passage stream, the more the stream will be cooled.

The main aim of the investigation was to determine the dimensions of the pond's cooling capacity for waste water. It was necessary to achieve such a streamflow distribution such that the passing stream will occupy as much as possible of the pond surface, reducing the vortex and stagnant zones to a minimum.

To solve the problem it was necessary to investigate the flow pattern of the discharge into the pond and, further, to the cooled-water intake. Recommendations for the types, dimensions and location of the flow-guiding installations had to be established, with the purpose of eliminating the stagnant zones and including them in the passing stream.

2. An old, dry branch of the Kuban' River bend served as a cooling pond. The bend has the shape of a horseshoe with an over-all length of approximately 7 km. An earth dike in the middle of the pond divides it into two almost equal parts — hot and cold. The ends of the horseshoe-shaped pond are connected by a channel drawing water from the hot into the cold part. The hot-water outlet and the cold-water inlet are placed on either side of the dike. The width of the pond in the hot branch is approximately 255 m, and in the cold branch it reaches 360 m. The width of the connecting channel is only 20 m.

The hot water is released through two chutes with discharges of 5 and 25 m^3/sec, which latter chute has been the object of the investigation.

In both branches of the pond the water enters in narrow streams into a considerably wide bed, which obviously creates vortex zones in these parts of the pond. In the hot branch the vortices are caused by the fast inflow of hot water through the chute and in the cold branch by the inflow of water from the relatively narrow channel into the wide bed of the pond, performing a 90° turn.

These two points of each branch determine, to a large extent, the behavior of the stream in the respective branch of the pond. The vortices created there spread out quite far down the stream, occupying large areas of the surface. The flow finds its way among them as a narrow stream.

The basic problems which were studied on two models were:

a) the flow pattern in the natural state of the pond, the flow path of the current and surface velocities of the entire area, position in plane of the passing stream, stagnation and vortex zones, and backstreams;

b) the influence of various current-distribution structures and the selection of such structures as will ensure maximum widening of the passing stream over the entire area of the pond;

c) the selection of such a design for the connection between the channel and the cold-water branch of the pond as will require the least amount of earthwork and the simplest current-structures for the best utilization of the cold branch of the pond;

d) the selection of a design for the outlet structure.

Different types of cantilevered, excavated, warm-water outlets have been studied in order to recommend the structure that will guarantee the quietest flow into the pond and the minimum funnel scouring.

3. The investigation was carried out on two three-dimensional hydrotechnical models. Model No. 1 represented the entire pond to a horizontal scale of 1:650 and a vertical scale of 1:130. Model No. 1 was built of concrete. Model No. 2 represented only the head part of the warm branch of the pond and served to study in detail that part of the pond where the warm water is being discharged.

This model, built to a 1:80 scale in a sandy bed, exactly simulated the types of outlet structures under study.

Since, under natural conditions, the current velocities in the pond are extremely small, it was impossible to establish a hydraulic similarity between model and prototype using normal Froude numbers, even with markedly distorted scale values of the model. A laminar flow always appeared on the model while actually the flow has been consistently turbulent. The laminar flow produced in the model a favorable pattern of uniform flow spreading over the entire surface of the pond, which will not occur in nature because of the turbulent flow. Therefore, in the hydraulic simulation, the method of exaggerating the discharge through the model was applied in order to produce a turbulent flow.

In increasing the discharge on the model, two conflicting requirements must be satisfied:

a) the discharge must be large enough to create a turbulent state of flow in the zone studied;

b) the discharge increase must not be excessive, otherwise it will distort the hydraulic pattern in places of abrupt change of the state of flow because of changes from one flow type to another.

The changes in the state of flow were due to:

a) the water stream falling from a cantilevered outlet;

b) the narrow channel feeding the water into the cold branch of the pond and the stream turning at the outlet from the channel.

This required different discharges at different parts of the pond and necessitated the study of each part separately. The discharges used on the model have been 1.5 to 10 times larger than those required according to laws established by Froude.

4. For a better stream distribution and for elimination of vortex and stagnant zones, it is recommended, after studying a series of measures, to place screen partitions within some of the streams, thus changing the velocity distribution. A length of 530 m of simple screen walls has to be placed in order to draw into cooling activity 415,000 m^2 of stagnant zones.

The comparison between two cantilevered water outlets proved the better performance of a cantilever with a slotted, flow-dissipating structure (nose) developed by V. D. Zhuzhnev of the Stavropol' Branch of

Yuzhgiprovodkhoz, as compared with that of the Lengiprovodkhoz-type cantilever structure with a water-deflecting wall.

5. The study was performed at the request of the Rostov Branch of Teploelektroproekt. The results of the investigation were used in the construction of the Krasnodar thermal power plant.

PART IX

SUMMARY OF ADVANCED METHODS AND APPLICATION OF SCIENTIFIC FINDINGS

VNIIG IMENI B. E. VEDENEEV SCIENTIFIC-TECHNICAL INFORMATION SERVICE

Head: A. A. Pichuzhkin, Candidate of Technical Sciences, Senior Research Worker

PREPARATION AND PUBLICATION OF ANNOTATIONS TO THE SCIENTIFIC RESEARCH WORKS OF THE VNIIG FOR 1957

Responsible for Research: I. A. Girshkan, Candidate of Technical Sciences, Senior Research Worker

A printed collection of 102 annotations to the scientific research studies carried out during 1957 was compiled in 1958. It included annotations to studies carried out within the framework of the scientific research program of the Institute, and to the most important studies carried out on a contractual basis.

COMPILATION, EDITING AND PREPARING FOR PUBLICATION OF TECHNICAL-INFORMATION BOOKLETS

Responsible for Research: I. A. Girshkan, Candidate of Technical Sciences, Senior Research Worker

In 1958 the VNIIG continued to publish technical-information booklets on results of scientific research work carried out by the Institute in the field of design and operation of hydro structures.

The following papers were published:

Yorish, E. L. and V. A. Melent'ev. Perekrytie rek namyvom banketa pri postroike gidrouzlov. (Damming of Rivers by Erecting Earth-fill Dams Required by the Construction of Hydro Developments);

Rozanov, N. S. Metod tenzosetki i ego prilozhenie k issledovaniyu napryazhennogo sostoyaniya gidrosoouruzhenii (The Strain-gage Network Method and Its Use in the Study of the Stressed State of Hydro Structures);

Chistyakov, A. M. Novaya metodika model'nykh issledovanii turbin reaktivnogo tipa i gidroturbinnykh blokov GES (New Methods of Model Testing of Reaction and other Types of Hydraulic Turbines of HEPs);

Ginzburg, M. B. Opredelenic protivodavleniya v gravitatsionnykh plotinakh na skal'nom osnovanii (po dannym otechestvennykh i zarubezhnykh naturnykh issledovanii) (Determination of Uplift Pressure in Gravity Dams Erected on Rocky Ground) (Based on Data of Soviet and Foreign Field Investigations);

Gubar A. S. and V. V. Stol'nikov. Primenenie melkozernistykh peskov v gidrotekhnicheskom betone (The Use of Fine-grained Sand in Hydraulic Concrete);

Kravtsov, V.I. Oblegchennaya betonnaya vodoslivnaya plotina na skal'nom osnovanii (Lightweight Concrete Spillway Dam on Rocky Grounds);

Kravtsov V.I. Novyi tip glukhoi plotiny dlya osnovanii s razlichnoi stepen'yu podatlivosti (New Type of Nonoverflow Dam Erected on Ground of Varying Yield Strength);

Kravtsov, V.I. Nekotorye progressivnye printsipy proektirovaniya arochnykh plotin (Certain Advanced Principles in the Design of Arch Dams);

Chikvashvili B. M. Raspredelenie davlenii na vkhodnykh ogolovkakh vodosbrosnykh galerei i propusknaya sposobnost' ikh pri sovmetnir rabote s poverkhnostnym vodosbrosom (Pressure Distribution at the Inlet Heads of Water-outlet Galleries and Their Discharge Capacity when Operating Jointly with Spillway Outlets);

Sistematicheskii ukazatel' statei, napechatannykh v "Izvestiyakh VNIIG" za 1931-1958 gg (Systematic Index to Papers Published in the "Izvestya VNIIG" between 1931 and 1958) (Vol. 1-60), compiled by N. S. Kotlyarov, Junior Research Worker).

EDITING AND PREPARATION FOR PUBLICATION OF REPORTS DELIVERED AT THE SCIENTIFIC-TECHNICAL CONFERENCE ON HYDRAULIC ENGINEERING OF THE VNIIG, AND AT OTHER SCIENTIFIC CONFERENCES

Responsible Editor: I. A. Girshkan, Candidate of Technical Sciences, Senior Research Worker

In 1958 publication of all reports delivered at the scientific-technical conference of VNIIG was completed.

A series of reports delivered at the International Congress of Large Dams was also published.

The total number of published papers amounted to 55 printers' sheets.

SUMMARY OF FOREIGN EXPERIENCE IN DAMMING-UP LARGE RIVERS DURING THE CONSTRUCTION OF HYDRO DEVELOPMENTS

Responsible for Research: P. V. Samostrelov, Candidate of Technical Sciences, Senior Research Worker

Depending on the local geological and hydrological conditions at the construction site, availability of suitable construction equipment, and on the results of laboratory investigations, the following methods of erecting rock dikes and impervious cores are being applied at present:

1) hydraulic filling of the rock dike in flowing water;

2) placing of material for the rock dam into the river bed by cable cranes;

3) placing of material from dump trucks mounted on trestlework or from pontoon bridges, simultaneously along the whole length of the passageway;

4) placing of rocks from dump trucks into the flowing water from the ends of the cofferdams at one river bank, or simultaneously at both banks.

The paper describes the latter of these methods which has been used in dike construction on the hydro developments of Chief Joseph, Dallas, and Oahe in the U. S. A. and Ohakuri in New Zealand.

The damming of the Columbia River at the site of the Chief Joseph development took 7 days (1952), of the Missouri River at the site of the Oahe development, 21 hours (1958).

The pontoon method of damming has many advantages and is more economical than other, conventional methods.

SYSTEMATIC DESCRIPTION OF FOREIGN HYDROELECTRIC POWER PLANTS AND OTHER HYDRO DEVELOPMENTS AND THEIR EQUIPMENT

Responsible for Research: P. V. Samostrelov, Candidate of Technical Sciences, Senior Research Worker

A compilation of methodical descriptions of foreign HEPs (both existing and under construction) started at the VNIIG in 1951. Up to 1958 descriptions of fifty foreign HEPs and development projects were compiled. In 1958 the following descriptions were compiled:

1. Of the hydro development Busan on the Zezere River in Portugal. This hydro development consists of a 63 m high arch dam with a crest spillway, powerhouses mounted on the diversion channel, shaft-type water intake, penstocks, bottom sluice gates, and a step-up transformer station. The capacity of the HEP is 56,000 kw.

2. The following run-of-river plants built on navigation canals:

a) Montélimar on the Rhone River (France) with a power output of 270.000 kw;

b) Fessenheim at the Alsace canal (France) 176.000 kw;

c) Wildegg-Brugg on the Aare River (Switzerland).

3. Buttressed dam Ben Metir (Tunisia), 71 m high.

4. Rock-fill dam Montgomery (U. S. A.), 34.3 m high with an asphalt-concrete facing (diaphragm) at the upstream slope.

The paper also contains a survey of hydro construction in Canada and of modern earth-fill dam construction in the U. S. A.

COMPILATION OF A LIST OF FOREIGN SCIENTIFIC RESEARCH INSTITUTES FOR HYDRAULIC AND HYDRO-ENGINEERING STRUCTURES

Responsible Editor: P. V. Samostrelov, Candidate of Technical Sciences, Senior Research Worker

This compilation is actually a continuation of the work started at the ONTI in 1956. In 1958 a report describing the work of the following institutions was prepared:

1. Hydraulic Laboratory at St. Anthony Falls (Minneapolis, U. S. A.);
2. National Hydraulic Laboratory Chatou (France);
3. Scientific Research Center of the Snowy Mountain Scheme (Australia);
4. Scientific Research Center of the U. S. Bureau of Reclamation, Denver;

5. Experimental Waterways Station of the Corps of Engineers, US Army;
6. Experimental Institute for Models and Structures (Italy).

These descriptions contain details of buildings, equipment and work programs, and include a bibliography of the most important works.

SPEEDING-UP CONSTRUCTION OF HYDROELECTRIC PROJECTS AND REDUCTION OF CONSTRUCTION COSTS

Responsible for Research: A. A. Pichuzhkin, Candidate of Technical Sciences, Senior Research Worker

The purpose of this study was to work out suggestions for speeding up the construction of hydro developments and for reduction of their construction costs.

Recommendations dealt with:

a) improvement in methods of calculation and of construction norms;
b) improvement of project layouts;
c) use of new types of hydro-engineering structures;
d) use of new types of building materials;
e) improved planning in construction work.

A considerable number of these suggestions was approved at the meetings of the Technical Council of the Ministry of Electric Power Stations held on 15 and 19 April 1958, and subsequently incorporated into the "Directives for Cost Reduction in the Design and Construction of Hydraulic and Electrical Equipment of HEPs"* published in 1958.

On the basis of these "Directives", construction-cost estimates for hydro plants under construction were revised.

The results of this work are being incorporated into the revised edition of "Technical Specifications and Norms", and are also taken into account in the drawing up of new reference material.

NEW AND REVISED TECHNICAL SPECIFICATIONS, NORMS AND INSTRUCTIONS RELATING TO HYDRO CONSTRUCTION

Responsible for Research: M. Ya. Krukovskii, Candidate of Technical Sciences, Senior Research Worker

The following departmental technical specifications, norms and instructions have been worked out and published:

1. Rukovodyashchie ukazaniya po proektirovaniyu na elektrostantsiyakh sistem gidrozoloudaleniya tsentral'nym apparatom Moskal'kova (Guiding Directives for the Design of Liquid-Slag-Removal Systems in Electric Power Plants by Means of the Moskal'kov Apparatus) (Laboratory for Hydraulic Mechanization; Author: E. Z. Nagli, Junior Research Worker);

* ["Ukazaniya po snizheniyu stoimosti pri proektirovanii gidroelektrostantsii v chasti gidrosooruzhenii i gidrosilovogo i elektrotekhnicheskogo oborudovaniya".]

2. Tekhnicheskie usloviya i normy proektirovaniya. Podzemnyi kontur plotiny na neskal'nom osnovanii. MES-125-57 (Technical Specifications and Design Norms for Underground Foundation Elements of Dams Erected on Rocky Grounds. MES-125-57) (Seepage Laboratory imeni N N. Pavlovskii; Author: Professor R. R. Chugaev);

3. Tipovaya instruktsiya po ekspluatatsii mekhanicheskogo oborudovaniya gidrotekhnicheskikh sooruzhenii (zatvory vodopropusknykh otverstii) (Standard Instructions on Operation of Mechanical Equipment for Hydro Structures) (Water-discharging Gates) (Laboratory for the Study of Operational Problems of HEPs; Author: S. M. Nalimov, Candidate of Technical Sciences, Senior Research Worker).

4. Instruktsiya po ispytaniyu prochnosti betona v sooruzheniyakh diskovym priborom DPG-4 (Instructions for Testing the Strength of Concrete in Structures by Means of the DPG-4 Device) (Concrete Laboratory; Author: V. B. Sudakov, Senior Engineer).

The following material has been prepared for publication:

1. Tekhnicheskie usloviya i normy proektirovaniya. Tsementatsiya v gidrotekhnicheskikh tunnelyakh. MES-161-57 (Technical Specifications and Design Norms for Cement Grouting in Hydro-engineering Tunnels. MES-161-157) (Laboratory for Planning of Construction Work; Author: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker);

2. Tipovaya instruktsiya po ekspluatatsii gidrotekhnicheskikh sooruzhenii derivatsionnykh gidroelektrostantsii (Standard Instructions for Operation of Hydro Structures of the HEPs Installed on Diversion Channels) (Laboratory for Studies of HEP Operation; Author: N. N. Yagodin, Senior Research Worker).

CONCRETE LABORATORY

Head: Professor V. V. Stol'nikov, Doctor of Technical Sciences

REVISION OF EXISTING STANDARDS FOR HYDRAULIC CONCRETE

Responsible for Research: Professor V. V. Stol'nikov, Doctor of Technical Sciences

The laboratory prepared and approved for publication the following material:

GOST 4760. Beton gidrotekhnicheskii. Metody ispytaniya betona (vzamen GOST 4780-49) (GOST 4760. Hydraulic Concrete. Methods for Testing Concrete (Replaces GOST 4780-49);

Tekhnicheskie ukazaniya po proektirovaniyu sostavov gidrotekhnicheskogo betona (vzamen GOST 4781-49) (Technical Directives for Design of Hydraulic Concrete Mixes) (Replaces GOST 4781-49);

The publication of this material will bring to a conclusion the long-range project of revision of Government Standards for Hydraulic Concrete (GOST 4795-49 and GOST 4781-49) published in 1949 by the Concrete Laboratory.

VNIIG IMENI B. E. VEDENEEV LABORATORY FOR PLANNING AND CONSTRUCTION WORK

Head: I. E. Kartelev, Candidate of Technical Sciences, Senior Research Worker

REVISION OF AND AMENDMENTS TO SECTION III OF THE TECHNICAL SPECIFICATIONS FOR CEMENT-GROUT WORK

Responsible for Research: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

Research by: L. N. Paronyan, Junior Research Worker

A directive issued by the Gosstroi recommends revision of Section III of the Technical Specifications for cement-grout work in tunnels and for cement groutings of fissured rocks.

The Section concerned has been revised both editorially and in its contents, incorporating the new regulations.

Thus, new regulations have been included for the use of cement-clay mortars, and for acceptance and approval of cement grouting. The parts relating to the use of cement-clay mortars were worked out on the basis of Soviet and foreign experience gained in the use of mixes (mortars) containing various plastifying and dispersing agents and applied in the construction of subways and hydro structures.

Recommendations were made for raising the efficiency of the injection operations (increase of grouting pressure, new methods of grout injection, appropriate choice of distance between injection holes, and sequence of operations).

TECHNICAL AID IN THE INTRODUCTION OF WELL-PLANNED TECHNOLOGICAL PROCESSES OF CEMENT GROUTING BY MEANS OF CEMENT-CLAY MORTARS, AND IMPROVED BATCHING OF SUCH MORTARS

Responsible for Research: A. N. Adamovich, Candidate of Technical Sciences, Senior Research Worker

Research by: L. N. Paronyan, Junior Research Worker

Fissures in the aleuritic and sand rocks underlying the foundation of the Irkutsk HEP had been sealed up by cement grouting. As far back as 1957 the VNIIG suggested the injection of cement-clay grouts containing surface-active admixtures into the grout curtains beneath the foundation of the power plant, and to subject the deep contact zones of the foundation to a special pressure treatment.

The construction laboratory [of the B. E. Vedeneev Institute] conducted special tests to ascertain the most suitable grout composition based on the inclusion of highly colloidal clays from local deposits. Appropriate instructions, based on the results of these tests, were passed on by the VNIIG to the Gidrospetsstroi which carried out the work. Thus, the institutes suggested to replace grout mixes having a water/cement + clay ratio of 3.6 commonly used for grouting soils of different specific water absorption, by three mixes with W/C+Cl ratio of 10.0, 8.0 and 4; this actually led to a better penetration of the grout in the rock pores.

The compositions were used according to their specific water-absorption coefficient and to the degree of fissuring of the rock layer.

Because of the fact that the results of cement grouting the contact zone between the rock layer and the foundation did not meet the specific requirements, it was decided to use compositions with a smaller clay content, which should have increased the strength of the contact zone.

Furthermore, cement additionally ground on a vibrating mill was injected with the grout into the contact zone. The duration of additional cement grinding was fixed by the Gidrospetsstroi at 2 hours. On the basis of available experimental data, duration of the additional vibro-grinding was reduced to 20 min Experience gained at the construction site points to the advantage of increasing the injection pressure from 10 to 12 atm. In the lower zones of the rock layer, the injection pressure was sometimes increased up to 16 atm. Final compacting was carried out at a pressure of 20 atm. Because of lack of high-pressure hoses, the injection pressure could not be increased further.

The increase of pressure by 20-25%, led to a marked improvement of cement grouting. The Institute representative, working on the improvement of grout composition in the construction laboratory, showed that the use of calcined soda for dispersion of fat clay (of the Kherdovsk deposits) is not desirable because it leads (in the presence of clay) to coagulation of the mix. Therefore, the use of $Na_2P_4O_7$ (sodium pyrophosphate) was suggested, because its use improves the flowability of the clay grout as much as $2\frac{1}{2}$ times compared with that obtained by the optimum batching of calcined soda. Furthermore, it has been established that sodium pyrophosphate even plastifies. In some cases the use of an optimum batch of sodium pyrophosphate of 0.15 to 0.20% of dry clay weight was suggested.

It was established that the use of additional grinding of cement on vibrating mills ensures full and effective dispersion of clay in the grout mix. Optimum duration of vibration grinding was found to be 15 to 20 min. Screen analysis of the ground products showed that sand fractions contained in clay in an amount of 5 to 8% were completely ground.

It was found that for a period of 1 to 1.5 days, clay underwent intense swelling amounting to 40 or 50% of its initial volume. The suggestion to use vibro-ground cement was implemented at the construction site. After one of the sections of the grout curtain was cemented, a number of test holes were drilled. Most of them showed that grout absorption by the rock layer met the requirements of technical specifications.

VNIIG IMENI B. E. VEDENEEV WATERPROOFING LABORATORY

Head: Professor P. D. Glebov, Doctor of Technical Sciences, Honored Scientist of the R. S. F. S. R.

TECHNICAL ASSISTANCE TO DESIGN AND BUILDING AGENCIES

Responsible for Research: S. N. Popchenko, Candidate of Technical Sciences, Senior Research Worker

Research Team: N. S. Pokrovskii, Candidate of Technical Sciences, Senior Research Worker
M. G. Staritskii, Candidate of Technical Sciences, Senior Research Worker
S. N. Popchenko, Candidate of Technical Sciences, Senior Research Worker

The purpose of the present work was to render technical aid to design and construction organizations in the use of asphalt linings, and hot and cold asphalt plaster-type waterproof layers.

1. Technical aid was rendered to Mosgiprotrans in the construction of asphalt concrete for protective linings of the Amu-Dar'ya River bank at the bridge crossing near the town of Chardzhou. A representative of the laboratory supervised the progress of work on the spot.

2. Technical aid was rendered to Trusts Nos 103 and 104 of Glavleningradstroi, and to Trust No. 1 of Lenotdelstroi in the use of cold asphalt plaster-type waterproof layers for various important structures.

3. Sample testing (in 1958) of 6 kinds of waterproofing material, the samples having been inserted in 1957 into the cooling towers of the Mosenergo and the Lenergo thermal-electric plants in the town of Slantsy. Investigations showed that after a period of one year inside the towers, the hot and cold waterproofing samples remained in good condition.

4. Technical aid was rendered to the trust Gidrospetsfundamentstroi in the construction of a vertical sewer (header) at the Bereznika Chemical Works, the sewer being lined with hot asphalt waterproof layers. In order to supervise experimental work connected with the technical-aid program, a representative of the laboratory visited the construction site.

MIIVKh IMENI V. R. VIL'YAMS SCIENTIFIC RESEARCH DIVISION

TECHNICAL SPECIFICATIONS AND NORMS FOR THE DESIGN OF HYDRO STRUCTURES ON IRRIGATION CANALS

Scientific Guidance: Professor P. I. Shipenko,

Authors: N. A. Ladygin, Candidate of Technical Sciences, Lecturer
S. I. Kobek
I. A. Vasil'eva

Scientific Consultant: Academician E. A. Zamarin (VASKhNIL)

Between the years 1957 and 1958, the Scientific Research Division of the MIIVKh has drawn up, for the Giprovodkhoz, Technical Specifications and Rules in order to rationalize and unify the hydro structures on irrigation canals, to eliminate waste in the use of building materials, and to introduce into practice advanced technical methods. The Technical Specifications included:

I. Classification of hydro structures on irrigation canals.
II. Calculation of structural loads and of design norms.
III. Hydro-engineering calculations of structures.
IV. Evaluation of geotechnical properties of soils.
V. Erection of hydro structures on loess soils.
VI. Calculation of foundations for hydro structures on irrigation canals.
VII. Hydraulic calculations of hydro structures.
VIII. Calculation of plain-concrete and reinforced-concrete hydro structures.

IX. Instruction concerning design of various types of hydro structures.

Part of the studies carried out in 1957 has been reproduced and distributed to construction and scientific research organizations for comments which will be taken into account in the next edition.

A brief description of the topics covered by the Chair of Hydraulic Engineering in 1958 follows:

1. Calculation of plain-concrete and reinforced-concrete hydro structures. The rated loads on hydraulic structures for irrigation canals were calculated according to Chapter II of "Nagruzki na sooruzheniya i raschetnye normy (Structural Loads and Design Norms).- Moskva, Giprovodkhoz.

Structures up to Class III inclusive are built of hydraulic concrete while those of Class IV and V are built of conventional concrete.

Since design data for the calculation of limiting state of structures are as yet not available, calculations should be made according to admissible or destructive loads. Shrinkage and creep of concrete as well as the effect of temperature fluctuations should not be taken into account. When calculating deformations and displacements, the area and gyration (inertia) moments of cross sections should be fully considered, taking into account compression and expansion of concrete. Reinforced-concrete structures, possibly in contact with water, should be checked for resistance to crack formation. Concrete used in reinforced-concrete structures should possess compression characteristics matching those of grades not below grade "150."

2. Instructions concerning design of various types of structures. For junction structures the use of wall drainage is recommended.

The relief of ground-water pressure by drainage makes it possible to reduce both the dimensions of the retaining walls and the thickness of the water-deflecting slabs (aprons), and to ensure greater stability of linings. Dimensions are given for the lining of supporting walls whose use markedly reduces consumption of building materials.

A special chapter deals with the use of precast reinforced-concrete slabs, of box-type structural elements and of pipe sections

Much attention is paid to the sealing of structures and contraction joints by means of foam-rubber material and bitumen mats. Instructions are given for the selection of apron types according to the elastic properties of the soil. For foundations consisting of stratified ground, erection of locks of the nonsectional dock type is recommended.

The type of regulating water-outlets (of the open, tubular or buffer-wall type) should be chosen with due regard to the local topography and available water head.

Apart from conventional materials, water siphons should also be made of precast prestressed reinforced-concrete tubes for a pressure of up to 10 atm. Asbestos-cement tubes resisting pressures of up to 10 atm, may also be used. As an innovation it is suggested that reinforced-concrete pipe runs be joined by the V. G. Popov type joints, and asbestos-cement tubes, by Borodin type joints - both types ensuring complete watertightness. The chapter describes designs of couplings for joining asbestos-cement pipes with the concrete heads of the intake and outlet sections of the siphons.

Cover printed in Israel at the Jerusalem Post Press, Jerusalem

OTS 63-11159